NEW ENGLAND WATER WORKS ASSOCIATION.

ORGANIZED 1882.

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This Association, as a Body, is not responsible for the statements or opinions of any of its members.

ADJOURNED MEETING.

PARKER HOUSE, BOSTON, MASS., FEBRUARY 8TH, 1893.

The following members and guests were present:

MEMBERS.

Everett L. Abbott, Civil Engineer, New York City; Charles F. Allen, Treasurer, Hyde Park, Mass.; Charles H. Baldwin, Boston, Mass.; George E. Batchelder, Registrar, Worcester, Mass.; Joseph E. Beals, Supt., Middleboro, Mass.; Nathan B. Bickford, Supt. Water Works, O. C. R. R., Boston, Mass.; William R. Billings, Taunton, Mass.; George Bowers, City Engineer, Lowell, Mass.; Dexter Brackett, Asst. Engineer, City Engineer's Office, Boston, Mass.; Arthur W. F. Brown, Registrar, Fitchburg, Mass.; Ernest H. Brownell, Brown University, Providence, R. I.; Fred. I. Chaffee, Chairman Executive Board, Providence, R. I.; George F. Chace, Supt., Taunton, Mass.; Charles E. Chandler, City Engineer, Norwich, Conn.; Freeman C. Coffin, Boston, Mass.; R. C. P. Coggeshall, Supt., New Bedford, Mass.; H. W. Conant, Supt., Gardner, Mass.; F. W. Clark, Clerk Chestnut Hill Reservoir, Boston Water Works, Brighton, Mass.; Henry A. Cook, Supt., Salem, Mass.; George K. Crandall, Asst. Engineer, New London Conn.; Lucas Cushing, Asst. Supt., Boston, Mass.; Edwin Darling, Supt., Pawtucket, R. I.; William G. Curtis, Asst. Biologist, Boston Water Works, Brighton, Mass.; Nathaniel Dennett, Supt., Somerville, Mass.; Elmer E. Farnham, Supt., Sharon, Mass.; B. R. Felton, City Engineer, Marlboro, Mass.; Z. R. Forbes, Asst. Supt., Brookline, Mass.; John R. Freeman, Civil Engineer, Boston, Mass.; Frank L. Fuller, Civil Engineer, Boston, Mass.; L. L. Gerry, City Engineer, Dover, N. H.; Albert S. Glover, Boston, Mass.; W. J. Goldthwait, Marblehead, Mass.;

J. A. Gould, Jr., Asst. Engineer, City Engineer's Office, Boston, Mass.; Frederick W. Gow, Supt., Medford, Mass.; Sherman E. Grannis, Supt., New Haven, Conn.; John L. Harrington, Cambridge, Mass.; John Harris, Commissioner, Waltham, Mass.; John C. Haskell, Supt., Lynn, Mass.; L. M. Hastings, City Engineer, Cambridge, Mass.; Louis E. Hawes, Civil Engineer, Boston, Mass.; Allen Hazen, Chemist, Lawrence, Mass.; Horace G. Holden, Supt., Nashua, N. H.; Willard Kent, Civil Engineer, Woonsocket, R. I.; Patrick Kieran, Supt., Fall River, Mass.; George A. Kimball, Civil Engineer, Boston, Mass.; Thomas C. Lovell, Supt., Fitchburg, Mass; Josiah S. Maxcy, Treasurer, Gardner, Maine; William McNally, Registrar, Marlboro, Mass.; Edward C. Nichols, Commissioner, Reading, Mass.; John H. Perkins, Supt., Watertown, Mass.; Edward H. Phipps, Supt., West Haven Water Co., New Haven, Conn.; Charles E. Pierce, Supt., East Providence, R. I.; Prof. Dwight Porter, Mass. Inst. Technology, Boston, Mass.; Waldo E. Rawson, Supt., Uxbridge, Mass.; Walter H. Richards, Supt., New London, Conn.; George J. Ries, Supt., Weymouth Centre, Mass.; A. H. Salisbury, Supt., Lawrence, Mass.; Arthur F. Salmon, member Water Board, Lowell, Mass.; William T. Sedgwick, Prof. Biology, Mass. Inst. Technology, Boston, Mass.; George A. Stacy, Supt., Marlboro, Mass.; Eugene S. Sullivan, Supt., Mystic Division Charlestown, Boston, Mass.; Edwin A. Taylor, Civil Engineer, Boston, Mass.; M. M. Tidd, Civil Engineer, Boston, Mass.; D. N. Tower, Supt., Cohasset, Mass.; C. H. Truesdell, Civil Engineer, North Grosvenordale, Conn.; W. H. Vaughn, Supt., Wellesley Hills, Mass.; Charles K. Walker, Supt., Manchester, N. H.; Robert J. Thomas, Supt., Lowell, Mass.; W. P. Whittemore, Supt., North Attleboro, Mass.; Frederick I. Winslow, Boston, Mass.; George E. Winslow, Supt., Waltham, Mass.; S. J. Winslow, Supt., Pittsfield, N. H.; E. T. Wiswall, Commissioner, West Newton, Mass.; E. R. Jones, Boston, Mass.; F. W. Shepperd, "Fire and Water," New York city; James M. Betton, Agt. H. R. Worthington, Boston, Mass.; A. H. Broderick, Chadwick Lead Works, Boston, Mass.; Edward L. Ross, Chapman Valve Mfg. Co., Indian Orchard, Mass.; F. H. Hayes, Dean Steam Pump Co., Holyoke, Mass.; H. N. Libbey and G. A. Taylor, Gilchrist & Taylor, Boston, Mass.; J. A. Tilden, Hersey Mfg. Co., South Boston, Mass.; Henry F. Jencks, Pawtucket, R. I.; F. E. Stevens and S. B. Adams, Peet Valve Co., Boston, Mass.; H. L. Bond, Perrin, Seamans & Co., Boston, Mass.; I. W. Dodge, Standard Thermometer Co., Boston, Mass.; E. J. Snow, Thomson Meter Co., Brooklyn, N. Y.; J. P. K. Otis and G. H. Carr, Union Water Meter Co., Worcester, Mass.; B. F. Polsey, and J. H. Eustis, Walworth Mfg. Co., Boston, Mass.; H. A. Gorham, secretary the George Woodman Co., Boston, Mass.; H. G. H. Tare, H. R. Worthington, New York city.

GUESTS.

W. F. Allen, Boston, Mass.; A. W. Barns, Fitchburg, Mass.; R. D. Chase, Boston, Mass.; James Daley, Uxbridge, Mass.; John Dexter, North Grosvenordale, Conn.; A. J. Fabens, Salem, Mass.; A. Prescott Folwell, Newark, N. J.; C. R. Felton, Brockton, Mass.; Charles Firth, W. K. Harrington, Newport, R. I.; D. A. Hartwood, Fitchburg, Mass.; J. A. Higgins, Providence, R. I.; J. H. Howland, Boston, Mass.; E. W. Kent, Woonsocket, R. I.; J. W.

Milne, Fall River, Mass.; L. S. Pope, Boston, Mass.; H. P. Quick, John F. Simmons, Hanover, Mass.; George R. Stetson, S. H. Taylor, New Bedford, Mass.; Richard J. Tingley, Pawtucket, R. I.; G. C. Whipple, Brighton, Mass.; George E. Williams, Boston, Mass.; W. T. Wyman, Boston, Mass.

After lunch was served, President Chace called the meeting to order. The Secretary read the following applications for membership:

RESIDENT ACTIVE.

John J. Cavanagh, Water Commissioner, Quincy, Mass.
Herbert T. Whitman, Water Commissioner, Quincy, Mass.
John Macnair, member Water Board, Lynn, Mass.
D. A. Sutherland, member Water Board, Lynn, Mass.
George C. Whipple, Biologist, Boston Water Works, Chelsea, Mass.
Leonard P. Kinnicutt, Worcester Polytechnic Institute, Worcester, Mass.
James H. Higgins, Supt. Meter Dept., Providence, R. I.
Charles H. Bartlett, Civil Engineer, Manchester, N. H.
R. H. Tingley, Civil Engineer, Pawtucket, R. I.

NON-RESIDENT ACTIVE.

John A. Berkey, President Little Falls Water Power Co., St. Paul, Minn.

On motion of Mr. Brackett the Secretary was directed to cast the ballot of the Association for the gentlemen named above, which he did, and they were declared elected members of the Association.

The attention of the members was then invited to a paper by Mr. Freeman C. Coffin, Hydraulic Engineer, Boston, on "Standpipes—their Construction and Design."

At the conclusion of Mr. Coffin's paper the President introduced Mr. A. Prescott Folwell, of Orange, N. J., who read a paper descriptive of a somewhat novel standpipe erected at Atlantic Highlands, N. J.

Mr. John C. Haskell, superintendent, Lynn, Mass., gave an interesting account of his experience with cement pipe. The discussion following was participated in by Mr. Jones of Boston, Mr. Winslow, of Pittsfield, N. H., and Mr. Darling of Pawtucket.

Mr. Walker of Manchester, N. H., then gave a short experience talk on "Fire Protection to Manufacturers."

On motion of the Secretary the thanks of the Association were tendered to Mr. F. S. Pierson, Chief Engineer of the West End Railway Co., for courtesies shown the Association to-day.

[Adjourned.]

During the forenoon many members made an excursion to the power station of the West End Railway Co., being shown about the works by Mr. F. S. Pierson, Chief Engineer of the Company.

QUARTERLY MEETING.

PARKER HOUSE, BOSTON, MASS., MARCH 8TH, 1893.

Previous to the formal meeting a very interesting exhibition of microscopic slides and photographs of different algæ and infusoria was made by Desmond FitzGerald, F. F. Forbes and Geo. F. Chace.

The following members and guests were in attendance:

MEMBERS.

Everett L. Abbott, Civil Engineer, New York City; Chas. F. Allen, Treasurer, Hyde Park, Mass.; Frank A. Andrews, Asst. Supt, Nashua, N. H.; Chas. H. Baldwin, Boston, Mass.; Albert P. Barrett, Registrar, Woburn, Mass.; Geo. E. Batchelder, Registrar, Worcester, Mass.; Joseph E. Beals, Supt. Middleboro, Mass.; Nathan B. Bickford, Supt. W. W. O. C. R. R., Boston, Mass.; Geo. Bowers, City Engineer, Lowell, Mass.; Dexter Brackett, Asst. Engineer, City Engineers Office, Boston, Mass.; George F. Chace, Supt., Taunton, Mass.; Chas. E. Chandler, City Engineer, Norwich, Conn.; Freeman C. Coffin, Civil Engineer, Boston, Mass.; R. C. P. Coggeshall, Supt., New Bedford, Mass.; Frederick W. Clark, Clerk, Chestnut Hill Reservoir, Brighton, Mass.; Byron I. Cook, Supt., Woonsocket, R. I.; Geo. K. Crandall, Asst. Engineer, New London, Conn.; P. F. Cully, Supt. Woburn, Mass.; Lucas Cushing, Asst. Supt., Boston, Mass.; Prof. Thos. M. Drown, Mass. Inst. Tech., Boston, Mass.; Horace L. Eaton, City Engineer, Somerville, Mass.; B. R. Felton, City Engineer, Marlboro, Mass.; F. F. Forbes, Supt., Brookline, Mass.; Desmond FitzGerald, Supt. Western Division Boston Water Works, Brookline, Mass.; Frank L. Fuller, Civil Engineer, Boston, Mass.; John R. Freeman, Hydraulic Engineer, Boston, Mass.; W. J. Goldthwait, Marblehead, Mass.; Richard A. Hale, Hydraulic Engineer, Lawrence, Mass.; Geo. W. Harrington, Supt., Wakefield, Mass.; John L. Harrington, Cambridge, Mass.; John Harris, Commissioner Waltham, Mass.; L. M. Hastings, City Engineer, Cambridge, Mass.; Clemens Herschel, Hydraulic Engineer, New York City; Horace G. Holden, Supt., Nashua, N. H.; James H. Higgins, Supt. Meter Dept., Providence, R. I.; Horatio N. Hyde, Supt., Newtonville, Mass.; Geo. A. Kimball, Civil Engineer, Boston, Mass.; E. W. Kent, Civil Engineer, Woonsocket, R. I.; Eugene P. LeBaron, Chairman, Middleboro, Mass.; Wm. McNally, Registrar, Marlboro, Mass.; Albert F. Noyes, City Engineer, West Newton, Mass.; Edward H. Phipps, Supt., New Haven, Conn.; Walter H. Richards, Supt., New London, Conn.; Geo. J. Ries, Supt., Weymouth Centre, Mass.; Henry W. Rogers, Supt., Haverhill, Mass.; Edward W. Shedd, Civil Engineer, Worcester, Mass.; Geo. A. Stacy, Supt., Marlborough, Mass.; Edwin A. Taylor, Civil Engineer, Boston, Mass.; Jos. G. Tenny, Supt., Leominster, Mass.; Robert J. Thomas, Supt., Lowell, Mass.; M. M. Tidd, Hydraulic Engineer, Boston, Mass.; D. N. Tower, Supt., Cohasset, Mass.; W. H. Vaughn, Supt., Wellesley Hills, Mass.; Chas. K. Walker, Supt., Manchester, N. H.; Geo. C. Whipple, Biologist, Boston Water Works, Brighton, Mass.; Horace B. Winship, Civil Engineer, Norwich, Conn.; Geo. E. Winslow, Supt., Waltham, Mass.; E. T. Wiswall, Commissioner, West Newton, Mass.; E. Worthington, Jr., Civil En-

gineer, Boston, Mass.; Richard R. Yates, Supt., Northboro, Mass.; James M. Betton, H. R. Worthington, Boston, Mass.; A. H. Broderick, Chadwick Lead Works, Boston, Mass,; F. H. Hayes, Dean Steam Pump Co., Holyoke, Mass.; Chas. H. Eglee, Contractor, Flushing, N. Y.; Geo. A. Taylor, Gilchrist & Taylor, Boston, Mass.; Henry F. Jenks, Pawtucket, R. I.; S. B. Adams and F. E. Stevens, Peet Valve Co., Boston, Mass.; H. L. Bond, Perrin, Seamans & Co., Boston, Mass.; H. H. Kinsey, Rensselaer Mfg. Co., Troy, N. Y.; H. C. Folger, Thomson Meter Co., Brooklyn, N. Y.; W. H. Moulton, Supt. Union Water Meter Co., Worcester, Mass.; Geo. B. Wood, R. D. Wood & Co., Philadelphia, Penn.; H. A. Gorham, The Geo. Woodman Co., Boston, Mass.

GUESTS.

Richard B. Allen, Lowell, Mass.; Hon. J. C. Brock, New Bedford, Mass.; J. W. Cassidy, Lowell, Mass.; D. W. Darling, Worcester, Mass.; Henry T. Fantom, Rockland, Mass.; Geo. W. Fuller, Lawrence, Mass.; Herman Gregg, Waltham, Mass.: Chas. T. Main, Civil Engineer, Boston, Mass.; W. F. Murphy, Brighton, Mass.; A. B. Rice, Boston, Mass.; Braddish J. Smith, New York City; F. L. Taylor, Brookline, Mass.; S. H. Taylor, New Bedford, Mass.

The Secretary presented the names of the following named candidates for active membership :

RESIDENT ACTIVE.

William E. Foss, Asst. Engineer Boston Water Works, 10 Fairview street, Dorchester, Mass.

George W. Fuller, Biologist, Lawrence Experimental station, Lawrence, Mass.

Joseph K. Nye, Fairhaven, Mass.

Caleb M. Saville, Civil Engineer, 44 Clarenden street, Malden, Mass.

Henry T. Fantom, Rockland, Mass.

NON-RESIDENT ACTIVE.

Watson L. Bishop, Superintendent Water Works, Dartmouth, Nova Scotia.

On motion of Mr. Brackett the Secretary was directed to cast the ballot of the association for the above named gentlemen, which he did, and they were declared elected.

On motion of Mr. Holden the president was instructed to appoint a committee of three to nominate officers for the ensuing term and to report at the June meeting. The president appointed Messrs. Horace G. Holden, Frank E. Hall and Charles K. Walker as the committee.

Mayor Brock of New Bedford was then presented by President Chace, and he briefly addressed the association.

The regular programme for the afternoon was then taken up, and Mr. L. M. Hastings, City Engineer, Cambridge, Mass., read a paper entitled "Description of a Method of Estimating the Loss of Water Power in a Stream by Taking Water therefrom for a City Supply." The paper was discussed by Mr. Herschel, Mr. Freeman, Mr. Kimball, Mr. Hale and Mr. Charles T. Main.

Mr. George A. Stacy, of Marlboro, and Mr. F. L. Fuller, of Boston, presented brief experience papers, and after the president had expressed his thanks to the members for the interest they had taken in the winter meetings and expressed the hope that he would see a full attendance at the annual meeting in Worcester, the association adjourned.

A METHOD OF ESTIMATING THE LOSS OF WATER POWER IN A STREAM BY TAKING WATER THEREFROM FOR A CITY SUPPLY.

BY

L. M. HASTINGS, CITY ENGINEER, CAMBRIDGE, MASS.

Perhaps one of the most uncertain and vexatious questions which comes to a community or corporation proposing to take water from a source which is, or may be, used for private purposes, is that of the damages which may be obtained by the owners to be affected by such taking. The cost of erecting the necessary works, piping, reservoirs, etc., can be estimated with tolerable accuracy, and the money expended for them has a tangible equivalent whose utility is apparent, but the amount of money to be obtained by the interested parties in the source from which the supply is taken is never easily estimated in advance, and seldom satisfactorily adjusted after the taking has been made.

Large sums are often expended in settlement of such claims for which little can be shown the inquiring taxpayer or stockholder.

The settlement of such claims has proved a very fruitful source of cases heard by commissioners or in the courts, for I think it is quite generally the experience that it is seldom that a settlement can be made with owners except by reference to such tribunals.

While it is impossible to ascertain precisely the number of such cases heard during the last 15 years in the state of Massachusetts for instance, it is certain that the number must be very large, and will be likely to increase as the introduction of public supplies become more general in the smaller communities of the country, and as the capacity of the old supplies are exceeded by the demands upon them and are sought to be reinforced with additional supplies.

In the preparation of these cases for hearing some very interesting problems in hydraulic and water supply engineering are often raised.

Engineers of high standing, engaged by the contesting sides are pitted against each other and sometimes the most surprising differences in results are reached.

While in some cases there may be local conditions and peculiarities which must be considered in estimating the value of the supply, yet there is in all the cases certain facts and principles which must be ascertained and applied in obtaining that criterion of value by which most of the sources of water supply are measured, viz: the *capacity of the stream for work* as measured in *average daily horse power*.

The amount of water and its value as used in manufacturing—other than as power—is a separate question and will not be considered at this time.

As an interesting case illustrating the methods which may be followed in estimating the power of a stream and the loss by taking from it, I propose to give as briefly as I can, some of the points in a suit brought by the owners of certain mills and water power on Charles river in Waltham and Wa-

tertown against the city of Cambridge, for the loss of water power caused by taking the water of Stony Brook for an additional water supply.

Since 1856 Fresh Pond has been a source of supply for the city. This supply proving inadequate, after various attempts to secure an additional supply, in 1885 the right was obtained from the legislature of the state to divert and use the water of Stony Brook and its tributaries in the city of Waltham and the towns of Weston and Lincoln for an additional supply supplementing that to be obtained from Fresh Pond.

The works were completed and the water let on from the Stony Brook reservoir in the fall of 1887.

After the settlement of various suits for loss of water power on Stony Brook itself, the owners of the four powers on Charles river, below its junction with Stony Brook, brought suit as I have said for compensation for loss of power in the river by the diversion of the water of Stony Brook its tributary.

The two upper dams at Waltham were owned by the Boston Manufacturing Company, the third dam, at Watertown, was owned by the Ætna mills, and the lower dam at Watertown Centre, was controlled by three parties jointly, the Watertown grist mill, Walker, Pratt & Co., foundry, and Hollingsworth & Whitney, paper manufacturers.

The claim made by each of the parties was the same in principle, although the drainage area was slightly increased at each successive dam, no notice was paid to the fact, and their claim was made proportionally to the head or fall at each dam.

Their statement of the case was very simple. It had been agreed by both sides that the yield of the 23 square miles of water shed of Stony Brook would be in an average year about 22½ million gallons daily.

Now this flow of 22½ million gallons daily, passed over the dam at Waltham of the Boston Manufacturing company with a fall of 12.10 feet, and 80 per cent. efficiency at the wheels would produce 90.6 horse power. This at the valuation placed upon a horse power by the agent of the company of $800 per horse power, gave a loss of $72,480. At the same company's lower or "Bleachery" dam, having a fall of 4.00 feet, a loss of 30 horse power on $24,000 was claimed.

At the Ætna mills dam, with a fall of 4.80 feet a loss of 36.0 horse power or $28,800 was claimed. At the Watertown dam having a fall of about 5.80 feet a loss of 43.40 horse power or $34,720 was claimed. making a total loss of 200 horse power valued by the owners at $160,000.

The city's defense against this claim was that but little and practically no loss of horse power was occasioned by the taking of such water of Stony Brook as the city could use, for the reason that in the seasons of the year when large quantities of water was flowing in Stony Brook, vast quantities of water were being wasted over the dams on Charles river owing to lack of pondage sufficient to retain the water until it could be used, and that when the seasons of lower flow came on, when the flow of the brook could be *used* and *not wasted*, the flow itself became so small as to be of little value.

Also it was sought to be shown that water power itself has but very little value at the present time any way.

This latter, an exceedingly interesting question, I shall not touch upon today, but shall try and explain how the former was attempted to be shown to the commission.

The drainage area is an important element in determining the power to be derived from any stream.

Charles river has its source in the town of Milford, Mass., from there flowing by a very circuitous route in a general north-easterly direction to the sea. On its way as it passes the town of Dedham it is tapped by a brook, partly natural and partly artificial, formerly called "Mill Creek" but now called Mother Brook, which flows from the Charles river into the Neponset river. This connection between this old "Mill Creek" and Charles river is by a ditch or canal through the meadows dug in 1639 at "common charges" as the quaint old records say. The digging of the ditch and consequent diversion of water from Charles river caused much litigation amongst the mill owners which went on until 1831, nearly 200 years after, when an agreement was signed by the mill owners on the two streams by which one-third of the water in Charles river at this point was to be diverted into Mother Brook and the remaining two-thirds was to continue to flow in the river, so that the drainage area of Charles river at any point below its junction with Mother Brook must be reduced by one-third the area above, or 66.64 square miles. The entire drainage area of Charles river above the dam of the Boston Manufacturing company is 251.42 square miles, deducting the 66.64 square miles above referred to, gives 184.78 square miles available drainage area of the river at this point.

No better data concerning the yield to be expected from a water shed in this vicinity can be obtained than the records given in the reports of the Boston Water Board of the observed yield of the basins of that system. These records were used in the computations which follow.

Another important element of water power is *storage capacity* or more properly "*pondage.*" Using the word storage to mean capacity to store or carry over a quantity of water from a wet season to a dry one and "pondage" the capacity to hold the night flow for use in the working hours of the following day. This last is the most usual method.

The amount of pondage exerts a very marked influence on the development of water power from the natural flow of a stream.

Thus in the case of the Boston Manufacturing company if they had no pond and took simply the natural flow of the stream as it came, but 289 horse power would be obtained from the river. With the pond which they have 503 horse power can be obtained, while with a pond large enough to retain the entire night and Sunday flow, about 809 horse power could be obtained. The amount of pondage depends not only on the *size* of the pond but on the elevation to which the water can be raised or lowered It is desirable, of course, to keep the water level as high as possible. If the draught, or low water line is not fixed by any riparian rights, then the *economical limit* must be determined by computation, for there will always be a point below which the gain in volume of water to be used will be offset by the resulting loss of head, as the pond is lowered.

The pond of the Boston Manufacturing company is the surface of the river extending from their dam to near the dam at Newton Lower Falls, a distance of nearly 5 miles, giving a surface area of about 10,750,000 square feet. The water can be drawn down at the dam about 2 feet below the flood line or level giving a pondage equal to 22,400 cubic feet per min. to be used in addition to the natural flow of the stream. This 22,400 cubic feet to be held back and stored in the pond during the 14 hours in which the mills are not running, and used in the 10 hours of the day when the mills are running.

Having the other elements determined, the power will vary directly as the head or fall of the stream. This can only be determined by careful measurements on the ground.

In order to present as clearly as possible just the power of the river and the effect on that power by abstracting the water of Stony Brook from it, the following exhibit, slightly condensed from the original, was made to the commissioners.

EXHIBIT NO. 1.

Showing Horse Power of Charles River at the Dam of the Boston Manufacturing Company in Waltham.

	I.	II.	III.	IV.	V.	VI.	VII.	VIII.
	Rainfall Collectable.	Amount collected and flowing off per month. (Area 184.78 square miles)	Amount flowing off per minute.	Amount of flow available for power in river.	Amount of flow not available for power.	Amt. of pondage. (To be added to amt. in col. IV.)	Gross amount available for power.	Power with effective head of 11. feet. 75 per cent. efficiency at the wheels.
	Ins.	Cubic feet.	Cubic feet per minute					Horse Power
January...	2.005	860,700,000	19,281	18,000	1,281	22,400	40,400	630.24
February .	3.206	1,376,260,000	33,831	26,000	7,831	22,400	48,400	755.04
March ..	4.997	2,145,100,000	48,053	26,000	22,053	22,400	48,400	755.04
April	3.609	1,549,260,000	35,862	26,000	9,862	22,400	48,400	755.04
May.	1.987	852,970,000	19,107	18,000	1,107	22,400	40,400	630.24
June.	0.864	370,894,500	8,585	8,000	585	14,154	22,154	345.64
July	0.335	143,807,000	3,221	3,000	221	5,266	8,266	128.90
August....	0.549	235,670,000	5,279	5,000	279	8,777	13,777	214.90
September	0.457	196,180,000	4,541	4,500	41	7,961	12,461	194.40
October...	1.053	452,030,000	10,126	9,500	626	16,677	26,177	408.36
November	1.617	694,130,000	16,068	15,500	568	22,400	37,900	591.24
December.	1.940	832,790,000	18,656	18,000	656	22,400	40,400	630.24

12)6,039.28

Average daily horse power for the year = 503.27

EXHIBIT NO. 2.

Showing Horse Power of Charles River at the Dam of the Boston Manufacturing Co., in Waltham after Diverting all the Water of Stony Brook.

	I.	II.	III.	IV.	V.	VI.
	Rainfall collectable.	Amount flowing off area 161.78 square miles.	Amount of flow in river available for power.	Amount of pondage to be added to column III.	Gross amount available for power.	Power with effective head of 11 ft. 75 per cent efficiency at the wheels.
	Inches.	Cubic feet per minute.				Horse Power.
January	2.005	16,871	15,750	22,400	38,150	595.14
February	3.206	29,600	22,750	22,400	45,150	704.34
March	4.997	42,046	22,750	22,400	45,150	704.34
April	3.609	31,379	22,750	22,400	45,150	704.34
May	1.987	16,718	15,750	22,400	38,150	595.14
June	0.864	7,512	7,000	12,385	19,385	302.41
July	0.335	2,818	2,275	4,958	7,233	112.83
August	0.549	4,617	4,375	7,680	12,055	188.06
September	0.457	3,973	3,938	6,966	10,904	170.10
October	1.053	8,860	8,313	14,592	22,905	357.32
November	1.617	14,060	13,563	19,600	33,163	517.34
December	1.940	16,324	15,750	22,400	38,150	595.14

12)5,546.50

Average daily horse power for the year = 462.20

Horse Power of Charles River, including Stony Brook, (Exhibit No. 1.) = 503.27 H. P.
Horse Power of Charles River, excluding Stony Brook, (Exhibit No. 2.) = 462.20 H. P.

Net loss of power as shown above = 41.07 H. P.

EXHIBIT NO. 3.

Showing Horse Power of Charles River allowed by the City of Cambridge as lost to the Boston Manufacturing Co. by taking the water of Stony Brook.

		IV.	V.	VI.	VII.	VIII	IX.	X.
		Amount of flow in River available for power.	Amount of flow not available for power.	Amt. of flow wasted over dam as shown by Boston Mfg. Co.'s records.	Flow of Stony Brook 23 square miles.	Estimated loss for power in natural flow of river.	Estimated loss (Col. VIII) developed by pondage.	Net H. P. on fall of 11 ft. 75 per cent. effciency at at the wheels.
		Cubic feet per minute.						Horse Power
January.	Col's I., II., III. same as shown in Exhibit No. 1.	18,000	1,281	5,215	2,180	...	.....	.....
February		26,000	7,831	10,800	4,092	...	.....	.....
March		26,000	22,053	13,830	5,005	...	.....	.....
April		26,000	9,862	11,570	3,889	...	.....	.....
May		18,000	1,107	6,715	2,260	...	.....	.....
June		8,000	585	1,907	1,065	533	1,475	23.01
July		3,000	221	68	549	549	1,572	24.54
August		5,000	279	380	698	698	1,923	30.00
September.		4,500	41	891	652	652	1,805	28.15
October		9,500	626	1,069	959	959	2,642	41.22
November		15,500	568	2,294	1,700	850	2,449	38.21
December		18,000	656	4,353	1,989	..	.....	.....
							12	185.13

Allowed loss of horse power in Charles River by taking Stony Brook = 15.43

In Exhibit 1, *Column* I., shows the amount of water which may be expected to flow off from the drainage area in an average year and is given in inches of rainfall. This is based on the observed quantity flowing off from the adjacent water shed of the Sudbury river and is the average of many years observations.

Column II., is this amount of rainfall put into cubic feet per month from the area of water shed or 184.78 *square miles*.

Column III., is this amount reduced to cubic feet per minute.

Column IV., is based on the observed fact that in all streams it has been found impossible to utilize all the flow of the stream for power, but during storms and freshets a certain quantity will invariably be wasted.

This quantity wasted has never been accurately determined but is variously estimated at from 16 to 30 per cent. of the total yearly flow, mostly occurring in the three wet months of February, March and April.

It has also been found that it is impossible to design the wheel plant to take all the water as it comes—some water must be wasted or the wheels will work with a lower efficiency at times. In this column about 20 per cent. has been considered as wasted, which waste is shown in *Column* V.

Column VI. shows the amount of water in addition to the natural flow of the river shown in column IV., which may be held in the pond above the dam and used in the 10 working hours.

As will be noticed for the 5 months—June to November—all the flow of the river can be retained, and for the remaining 7, only the capacity of the pond, sufficient to give 22,400 cubic feet per min. can be held, the excess over this amount being wasted.

The total amount of water available for power shown in column VII. is reduced to horse power in column VIII. with a head of 11 feet—for while the whole head or fall is 12.10 feet, a certain amount of head is always lost in getting the water on and off the wheels, and also by the drawing down of the pond. Seventy-five per cent. of the gross power has been taken as the efficiency of first-class wheels.

The resulting daily horse power is 503.27 as shown.

To find the theoretical loss of power in the river by taking from it the water of Stony Brook the same method is followed, with a smaller water shed or drainage area, as shown in Exhibit No. 2. Instead of a drainage area of 184.78 square miles, we now substract the 23 square miles of water shed in Stony Brook, giving an area of 161.78 square miles.

The resulting power of the river is 462.20 horse powers. As the power of the river before taking out Stony Brook was 503.27, the difference, or 41.07 horse power would represent the calculated loss to the company. This would be *the most* that they could fairly claim and in a majority of cases if the entire flow of the stream was taken, would represent the loss of power in the river by such taking.

But as it would be a practical impossibility for *all* the water of the brook to be diverted, another exhibit was prepared which is shown here as Exhibit No. 3 and shows a somewhat different study of this case with additional data.

In this exhibit, columns I. to V. are identical with those shown in Exhibit No. 1.

For many years the Boston Manufacturing company have kept a record of the amount of water (depth in inches) flowing over their dam, observations being taken four times daily. Access to these records were kindly given me by the company, and the average amount wasted over their dam for a period of 10 years is shown in column VI.

The amounts estimated as wasted in column V. and that shown in column VI., while not precisely agreeing, yet show a strong general similarity in range, and were used as a study and comparison together, and with column VII. which gives the flow of the Stony Brook water shed of 23 square miles.

From a comparison of these three columns, the conclusion was drawn that during the months of January, February, March, April, May and December more water was wasted at the dam than the entire flow of Stony Brook.

This will be corroborated by a study of column IV., Exhibit 2, where it will be noted that the pond has not capacity to hold during the months above named, the night flow, even after the substraction of Stony Brook.

During the months of June and November it was allowed that one-half the flow of Stony Brook would be lost, and the entire flow during July, August, September and October as shown in column VII. This volume, developed by pondage is shown in column IX. and reduced to horse power in column X.

This reduced to a daily average for the year gives 15.43 horse power as the allowed loss at this dam.

During these months therefore it was claimed that no loss to the company would result by taking the water of Stony Brook, this taking simply reducing the amount of waste. Indeed it might be claimed that this would be a benefit rather than an injury by lowering the back water below the dam.

Associated with me on this case were Mr. N. Henry Crafts and Mr. Richard A. Hale. Both reached substantially the same results by somewhat different methods. Mr. Crafts assumed that as the mills claimed, and were fitted up for but 500 horse power there was no loss until the power fell below that amount. His estimate of loss was 14.71 horse power. Mr. Hale took the water required to give full power on the wheels and when there was not enough water to give full power computed the difference with and without Stony Brook, reaching substantially the same result.

On the basis of 15.43 horse power being lost at the upper dam the total loss at the four dams would be 34.33 horse power instead of 200 horse power as claimed.

Instead of a damage of $160,000 as claimed, the commissioners awarded as the total damage at the four dams, the lump sum of $15,000, which award was accepted by both parties.

DISCUSSION.

MR. HERSCHEL. I desire to call the attention of the gentlemen present to one feature of this matter of the taking of the flow of brooks and rivers for the domestic supply of cities and towns; and of the consequent questions of damages to mill owners for the so-called diversion of water. This whole subject cannot be separated from the legal aspects of the case, and a great deal of confusion is often caused in these hearings by lack or harmony between the labors of the engineers and of the wants and silent desires of the referees.

Nor need we be repelled by a seeming abstruseness of the points I am about to mention, merely because they are a part of the law. No subject, as has been said, is so abstruse but that an able presentation of it will make it plain. By dint of frequent association with lawyers in these cases, and hearing their able presentation of them, and from the decisions in such cases, I have gathered one simple proposition, which must be the goal in all valid computations and arguments in such estimations of damages

The closer it is kept in view, the less waste time will be included in the presentation of a case, and referring particularly to the labors of the engineers or other experts, the more valuable will they become in the determination of the award.

This simple proposition is, that the sum to be determined is the difference between the market value of the property under consideration, as it was before the water was diverted, and its market value after the water has been diverted. This is the question, one in simple substraction, which the referees or the jury are called upon to answer. Of course, before this sum in simple substraction can be performed, the referees or the jury have got to arrive at the minuend and at the subtrahend; but any work not bearing directly on the determination of these two, is really not relevant to the work in hand.

I am satisfied that it would be an aid to engineers and to others engaged in these cases in an honest, pains-taking way, to have this idea of market value presented to them, and kept before them in their work. And I am also satisfied that it is a lack of that consideration in the minds of all the parties concerned, which produces so wide variances between claims and final awards, as for instance that found in the case now under consideration; namely, as 160 is to 15; or between claims and admissions.

Referring now directly to the paper of Mr. Hastings, I desire to comment on the columns of the tables which bear on the question of what constitutes freshet water. This is always one of the interesting points in cases of this sort. My own conclusion has been, that on rivers which have been used for water power many years, especially on small rivers and brooks, this question has been answered in the course of years by the experience of the mill owners located upon it. At each mill privilege it has been quite well determined how much wheel capacity it will pay to set up, and whether it would or would not be prudent, and showing lack of business capacity to increase it. Of

course, this supposes no hard and fast rule; nor could this test be applied to a single owner of a single mill privilege, but there are usually a series of such mill privileges within a short distance of each other, on the same stream, each mill privilege held through a series of years, by many owners, of both phlegmatic and of sanguine temperaments, and from the result of their combined experience, as regards a proper and reasonable wheel capacity to maintain on that river, at that point, fully as close a determination can be made, of the quantity which constitutes a proper wheel capacity, and that other quantity which constitutes freshet water, as can be made of any other of the prominent data in the case. It is merely applying the all pervading principle of the "survival of the fittest" to water wheels. On most any river, some mill owner at one time or another, will have run away with the idea that he could get more power out of his property than anybody else could; upon which hard experience will soon have taught him, that it did not pay to have wheels and water connections stand idle so great a part of the year; and the final result will have been an abandoned water wheel. Other mill owners will have failed to fully utilize their water power property; but the concensus of opinion in this regard, among a series of owners during a series of years on the same site, and as confirmed or modified by other series of owners, up stream and down stream from the property in question, is a safe guide to follow.

In this way, I have observed, that very rarely will we find, on any such streams, a wheel capacity greater than would be able to run at full gate, for half the days of the year, the balance of the water during that time wasting over the dam as freshet water; with the wheels taking what they can get, the other half the days of the year. I do not refer to consecutive days of the year, but ranging the flow of the river, on the 365 days of the year, in their order of size, to the first half of the days of the year thus arranged, which will be the 182 or 183 days of largest river flow for the half the days of the year first above mentioned.

More frequently, this limit of commercially practicable wheel capacity is found such, that the wheels have all the water they want, and to spare, for some 243 days of the year, and must needs take what they can get the remaining 122 days. And somewhere between these limits of 182, and of 243 days, as fixing that river flow, which in turn, fixes the commercially practicable and reasonable wheel capacity, to be put in at any mill privilege, is the dividing line, by means of which we are enabled to say, not only for New England streams, but on any mill stream, what constitutes the flow of the river, which may prudently be utilized, and what of necessity constitutes freshet water.

Mr. Freeman. Mr. President, I was very much interested in hearing Mr. Herschel's opening remarks, from the fact that this morning I found lying on my desk a decision lately rendered by the Supreme Court of New Hampshire in a very celebrated water power case, one concerning the water rights in Lake Winnipiseogee, and in looking through that report it struck me most forcibly that the keynote was precisely the same proposition with which Mr.

Herschel began his discussion, viz., that the whole question of damages was to be determined by the fair market value or what a reasonable purchaser would probably pay for the property before the act as compared with the fair market value after the act if it were fairly presented for sale.

As to these tables, I think the most interesting column to the hydraulic engineer is the one marked column VI., showing the actual waste which occurs even during those months of the year when the flow is small. I have seen several times estimates concerning suits for damages made up without any realization of the fact that during even the dry months of the summer there was a considerable proportion of waste. You have a summer freshet, not infrequently an August freshet for instance, during which the water comes down in great volume for a day or two, and during those few days the larger portion of that flood water is wasted; now if taking the record of flow you view the matter simply in the light of a monthly average, as shown in the first column of Mr. Hastings table you get the whole thing smoothed out, and spread over the month raising the general level and so it tends to exaggerate the yield of the stream.

Concerning this question of exaggerating the yield of the stream, it struck me as I listened to the presentation which has been made here, that if a man were to estimate the value of the water power with a view to putting in wheels, or expecting to obtain that power to drive his mill to the amount which has been figured here, he might meet with disappointment.

There are various approximations made in these estimates, nearly all are on the side of the mill owner. That is they all tend to magnify the power a little and the net result of all would be, perhaps, to make the amount of power as shown by the computation anywhere from 10 to 25 per cent. greater than the actual amount of power which could be realized in practice.

For instance, the length of the working hours is here taken as though the water were used for exactly 10 hours per day for six days in the week and was stored for the 14 hours intervening between each day's work.

Now, in point of actual fact such factories commonly run for five days in the week a little over ten hours and a half each day, and so gain a half holiday on Saturday afternoon.

In addition to that, there is always a few minutes' waste of water in starting in the morning and at noon; and above all there is the waste of water which occurs even in the best regulated establishments by the leak of the wheel gates at night. All these things are matters which occur in every day practice. We do not get ideal wheel gates or ideal dams which hold every drop of water which comes into the mill pond at night.

If one is estimating steam power, he adds a certain percentage for the coal which is used in banking the fires at noon and for the starting in the morning. So just in the same way there should be a certain percentage allowed for the unavoidable waste in connection with water power. The water power is used *more than ten hours* a day and the fact of running so as to gain the Saturday half holiday tends to accumulate water at the very end of the week when in many cases even the Sunday flow cannot be stored

So as to many other things in the tables. For instance, of estimating the amount of available storage by drawing down a long, narrow pond like that, and assuming that it can at any time be drawn down to the extent of two feet over the whole surface and that all of the water contained in that two feet can be utilized during the working hours. As men practically run water wheels, they seldom do so large an amount of storing as that.

Then there is another point, the percentage of useful effect for the turbine. Almost any turbine maker will guarantee to put in a wheel to furnish 85 per cent. or 80 per cent., and throwing off five per cent. it would seem that 75 per cent. was fair. But there are other things to be taken into consideration. Wheels are not always in first-class condition as practically used, there may be obstructions in the buckets of the wheel, or other little things about the wheel plant which cut down the percentage. But even suppose the wheels were in perfect condition, there is the fact that in utilizing a flow varying so widely, from 67 horse power as the minimum to 128 as the maximum, you are compelled a part of the time to run your wheel under very disadvantageous conditions. So I think that taking things by and as large an average efficiency of 70 per cent. would be a generous statement for the ordinary case.

MR. KIMBALL. I have been very much interested, Mr. President, in this paper. I think there is very little in our literature which pertains to this subject in the line that has been presented here this afternoon. I entirely agree with Mr. Freeman in saying that the writer of the paper has been very liberal towards the mill owners in his estimates, and I desire to call the attention of this association to the fact that in the first column, the one which is the basis of this whole table, are figures which have been collected from the reports of the Boston Water Works, which have been kept very carefully for a great many years; and I think every engineer certainly appreciates what a great value those figures are to the profession. It was about twenty years ago while an assistant to Mr. Herschel we were computing the amount of power, and we had no data of this kind, in fact no reliable data by which we could learn the amount of water collectable from a water shed. We could easily measure the water shed, but we could not determine how much of that water could be depended upon for power. I think that the engineers of the Boston Water Works are entitled to a great deal of credit for the figures that they have prepared, which can now be used by engineers in cases of this kind.

MR. HALE. As bearing a little on what Mr. Freeman has said with regard to running the wheels under different conditions, I would say that in working up the table according to the method which I used, I took the wheels as they actually existed at the time. The owners acknowledged that the water power had been developed to the best advantage, that they should never enlarge it any more, and their steam power amounts to about four times what the water power is. So I took their wheels as I found them running, applying the amount of water flowing to the wheels, and seeing under what conditions they could run their wheels to the best advantage, setting them at such points at the gates as would give them the best efficiency, and the computation was worked up in that way, assuming that special attention was paid to that.

Starting from the same figures which Mr. Hastings used in his table, I arrived at about the same results as he did, but from a rather different point of view, taking their plant as they had it, and seeing what was the best use that could be made of the water flowing in the river under the actually existing conditions. And it was pretty liberal to the mills, as we consider it concentrated into ten hours' flow, while without doubt, as Mr. Freeman suggests, there was a great deal of waste at other times which could not be stored.

MR. NOYES. I would like to ask Mr. Hastings if in making his computation he allowed for the water taken for water supplies from the drainage area above.

MR. HASTINGS. No sir.

MR. NOYES. And for which we have settled with the mill owners for quite a handsome amount. There is Brookline, Dedham, Newton and Wellesley.

MR. HASTINGS. No account was made of the reduction on account of water supplies.

MR. NOYES. It would amount to somewhere from five to eight million gallons.

MR. HASTINGS. In view of the whole area of the water shed it would be very small, hardly worth considering, I think.

MR. NOYES. The mill owners didn't think so.

MR. HASTINGS. I would like to say, with regard to what Mr. Freeman has said that in these computations the effort was made to be more than generous with the mill owners, and not as much water was allowed as waste as I should certainly recommend any one who set up a wheel plant to allow for. I should not think it would be wise to set up a plant for more water than would be shown by Mr. Francis' plan of taking the May flow as the maximum amount to be used. With regard to the hours in the day, that varies with different mills, and it seemed impossible to assume any other basis than an ordinary work of ten hours a day, and so that was taken. With regard to the draft on the pond with two feet depth of storage, I did not allow for two feet in depth from the whole pond, but two feet at the dam and tapering up the 5-mile reach, I think, to six inches or one foot in depth, I have forgetten which. Of course it would be impossible to draw the pond down in that length of time level, so we would have the natural hydraulic head, so the pond was not a full pond two feet deep its entire length.

THE PRESIDENT. We should be glad to hear from Mr. Charles T. Main.

CHAS. T. MAIN. I think that a few words of explanation as to the great difference in the amount of damage claimed and the amount awarded and the bearing which such amounts have upon the tables presented by Mr. Hastings may be of interest.

The amount of damages as estimated by experts for the plaintiff is usually obtained as follows :

They estimate the average total flow of the stream, including freshet water, and then say if this be taken away, or a portion thereof, it will be necessary to erect, maintain and run a steam plant, complete in itself, of the average power thus taken away, and that the damage is a sum of money which will erect and maintain and run such a plant forever. In this way they figure out enormous sums which usually are absurd.

The method of Mr. Hastings is a method of determining the *average* power which the mills have been deprived of, but it does not tell the whole story as to the manner of estimating the damages.

If the tables were made out showing the average of the dryest months for sixteen years, the next dryest, and so on, instead of the average of the calendar months there would be a greater variation shown in the power. If alongside of this could be seen the monthly average for each year arranged in order of dryness, there would be shown a still greater variation of power, and if finally we could see the daily flow we should discover something which completely overthrows the theory upon which the large amounts were figured out by the plaintiff. We should find that the flow of the river is so variable that it is necessary for a concern which must run constantly to maintain a steam plant of sufficient size to run their machinery independent of the water power.

If, therefore, it be necessary for them to maintain such a plant it is not necessary for the defendants to pay for erecting, maintaining and running a separate plant, and the only loss to such a concern is the extra coal which they must burn to make up for the power taken away and any other incidental increase in running expense which may be incurred.

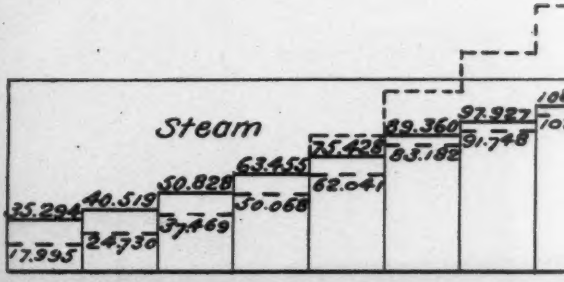
The effect of this upon the market value of the concern is this. If a man, desirous of purchasing such a concern, knew that a certain amount of water power were to be taken away soon after the purchase, he would ascertain jus what must be done to make this loss good and deduct such a sum from his estimate of value before taking. If he found that no additional plant would be required, but simply a small increase of coal consumption he would decrease the value accordingly.

Now it happens to be a fact that in the cases under consideration that those concerns requiring constant power were equipped with steam plants large enough to run their whole works, and the damages as estimated by the defendants were based upon this fact, together with some others, and consequently are much smaller than those of the plaintiff.

The other considerations perhaps have no reference to the paper under consideration, but do affect the market value of the property before and after the taking.

At the Ætna mills the steam plant was large enough to develop the whole power required, and they used all the exhaust steam from their present engines and could use more, so that the cost of producing the slight addition of power would be small.

Development of Steam o
for Dwight Printing Co., As
and after diversion of Whi
taking 1,563,000 cu. ft. as av
of Dwight Printing Co's mill
reckoning by months in or
yield.



Desmond FitzGerald
1893

and Water Power
Ashland before
hitehall Pond,
available storage
ill pond, and
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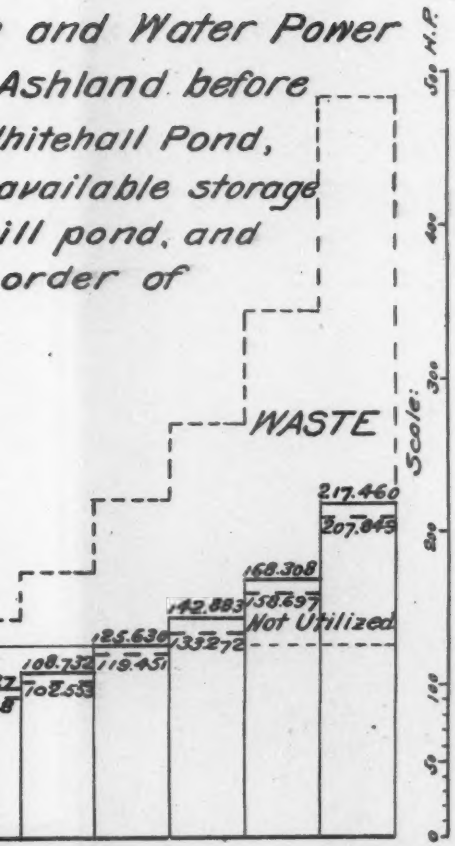

Available Flow and Power of the Dam of the Dwight Printing Company at Ashland; the pre

BEFORE THE DIVERSION OF THE WHITEHALL POND DRAINAGE AREA.

Months arranged in order of dryness.	Flow per calendar day. From the 20.56 sq. m. drainage area (natural flow). Cubic feet.	Flow per calendar day. From the Whitehall Pond drainage area (regulated flow. Cubic feet.	Flow per calendar day. From the total drainage area (20.016 sq. m.). Cubic feet.	Flow for 10 working hours from total area. Cubic feet.	Flow for 14 night hours from total area. Cubic feet.	One-sixth part of the Sunday (24 hour) flow that can be stored. Cubic feet.	Storage that can be added to 10 hour flow. Cubic feet.	Total flow from Dwight Printing Co.'s Mill Pond. Per day of 10 working hours. Cubic feet.	Per month of 85% working days of 10 hours. Cubic feet.	Per month of 30.44 calendar days of 24 hours. Cubic feet.	Flow per calendar month wasted because of insufficient storage capacity. Cubic feet.	Total flow from Mill Pond for 10 working hours reduced to net H. P. (1. e. 75 per cent.) on 18 ft. fall. Horse power.	Flo
1	502,516	483,117	985,633	410,680	574,953	164,272	739,225	1,149,905	29,514,228	30,000,000	486,000	35.29	F
2	690,623	"	1,173,740	489,060	684,680	146,387	831,067	1,320,127	33,883,260	35,726,000	1,843,000	40.52	(
3	1,063,670	"	1,546,787	644,490	902,297	110,117	1,012,414	1,656,904	42,527,203	47,080,000	4 553,000	50.86	1,(
4	1,518,360	"	2,001,477	833,950	1,167,527	65,912	1,233,439	2,067,389	53,062,984	60,920 000	7,857,000	63.46	1,5
5	1,950,440	"	2,433,557	1,013,980	1,419,577	23,904	1,443,481	2,457,461	63,074 832	74,072,000	10,997,000	75.43	1,9
6	2,753,049	"	3,236,166	1,348.400	1,887,766	0	1,563,000	2,911,400	74 725 933	98,501,000	23,775,000	89.36	2,7
7	3,422,870	"	3,905,987	1,627,490	(Only	0	"	3,190,490	81,889,243	118,889,000	37,000,000	97.93	3,4
8	4,267,780	"	4,750,897	1,979,540	1,563,000	0	"	3,542,540	90,925,193	144,605,000	53,680,000	108.73	4,2
9	5,589,060	"	6,072,177	2,530,070	can be	0	"	4,093,070	105,055,463	184,822,000	79,767,000	125·63	5,5
10	6,669,720	751,514	7.421.234	3,092,180	stored)	0	"	4,655,180	119,482,953	225,884.000	106,401.000	142.88	6.6
11	8,657,760	"	9,409,274	3,920,530		0	"	5,483,530	140,743,937	286.394,000	145,650,000	168.31	8,6
12	12,501,100	"	13,252.614	5,521,920		0	"	7,084 920	181,846,280	403,376,000	221,530,000	217.46	12,5

rinting Company at Ashland; the present storage is taken as 1,563,000 cubic feet and the net fall is taken as 13 feet.

DESMOND FITZGERALD, CIVIL ENGINEER.

DRAINAGE AREA.					AFTER THE COMPLETE DIVERSION OF THE WHITEHALL POND DRAINAGE AREA.										Effect of the total diversion of the flow of Whitehall Pond drainage area, the extent of probable development of water power being taken as 125.63 H. P. net. (9th No., Col. 13.)		
low from Dwight Printing Co.'s Mill Pond.					Flow from the 28.662 sq. m. drainage area.					Total flow from Dwight Printing Co.'s Mill Pond.							
	Per month of 25¾ working days of 10 hours. Cubic feet.	Per month of 30.44 calendar days of 24 hours. Cubic feet.	Flow per calendar month wasted because of insufficient storage capacity. Cubic feet.	Total flow from Mill Pond for 10 working hours reduced to net H. P. (L. 75 per cent.) on 13 ft. fall. Horse power.	Per calendar day of 24 hours (natural flow). Cubic feet.	For the 10 working hours. Cubic feet.	For the 14 night hours. Cubic feet.	One-sixth part of the Sunday (24 hour) flow that can be stored. Cubic feet.	Storage that can be added to 10 hour flow. Cubic feet.	Per day of 10 working hours. Cubic feet.	Per month of 25¾ working days of 10 hours. Cubic feet.	Per month of 30.44 calendar days of 24 hours. Cubic feet.	Flow per calendar month wasted because of insufficient storage capacity. Cubic feet.	Total flow from Mill Pond for 10 working hours reduced to net H. P. (L. per 75 cent.) on 13 ft. fall. Horse Power.	Amount of Water Power developed before diversion. Horse power.	Amount of Water Power developed after diversion. Horse power.	Difference, or amount of Water Power lost by diversion. Horse power.
,905	29,514,228	30,000,000	486,000	35.29	502,516	209,380	293,136	83,753	376,889	586,269	15,047,571	15,295,000	247,000	17.99	35.29	17.99	17.30
,127	33,883,260	35,726,000	1,843,000	40.52	690,623	287,760	402,863	115,104	517,967	805,727	20,680,326	21,021,000	341,000	24.73	40.52	24.73	15 79
,904	42,527,203	47,080,000	4,553,000	50.86	1,063,670	443,200	620,470	157,088	777,558	1,220,758	31,332,789	32,375,000	1,042,000	37.47	50.86	37.47	13.39
,389	53,062,984	60,920,000	7,857,000	63.46	1,518,360	632,650	885,710	112,882	998,592	1,631,242	41,868,545	46,215,000	4,346,000	50.07	63.46	50.07	13.39
,461	63,074,832	74,072,000	10,997,000	75.43	1,950,440	812,680	1,137,760	70,873	1,208,633	2,021,313	51,880,367	59,367,000	7,487,000	62.04	75.43	62.04	13.39
,400	74,725,933	98,501,000	23,775,000	89.36	2,753,049	1,147,100	1,605,949	0	1,563,000	2,710,100	69,559,233	83,796,000	14,237,000	83.18	89.36	83.18	6.18
,490	81,889,243	118,889,000	37,000,000	97.93	3,422,870	1,426,200	(Only	0	"	2,989,200	76,722,800	104,184,000	27,461,000	91.75	97.93	91.75	6.18
,540	90,925,193	144,605,000	53,680,000	108.73	4,267,780	1,778,240	1,563,000	0	"	3,341,240	85,758,493	129,900,000	44,142,000	102.55	108.73	102.55	6.18
,070	105,055,463	184,822,000	79,767,000	125.63	5,589,060	2,328,770	can be	0	"	3,891,770	99,888,763	170,117,000	70,228,000	119.45	125.63	119.45	6.18
,180	119,482,953	225,884,000	106,401,000	142.88	6,669,720	2,779,050	stored.)	0	"	4,342,050	111,445,950	203,010,000	91,564,000	133.27	"	125.63	0
,530	140,743,937	286,394,000	145,650,000	168.31	8,657,760	3,607,400		0	"	5,170,400	132,706,933	263,520,000	130,813,000	158 70	"	"	0
,920	181,846,280	403,376,000	221,530,000	217.46	12,501,100	5,208,790		0	"	6,771,790	173,809,277	380,502,000	206,693,000	207.85	"	"	0

At the Waltham Bleachery there is a fall of only four feet. This has not been developed for power and no one would think of doing so at the present day. The loss to them of *power* is therefore nothing.

At Hollingsworth & Whitney's the use of water power has been abandoned oweing to the fluctuation of power and other reasons.

The powers at Perkins' grist mill and Walker, Pratt & Co., are not only affected by the variable flow of the stream but also by the tide, thus making an extremely variable power. How such power can be considered of equal value to a constant steam power I cannot conceive.

After having obtained the average power which is taken away it is necessary to know the variation in the flow of the stream and the local conditions for each property which is deprived of such power before it can be determined what the approximate decrease in market value would be.

MR. FITZGERALD. Mr. Hastings' paper has interested me very much. It has been my lot to have been engaged in a number of similar cases in the past few years. While no case arises in which something is not to be learned yet the figures of damages are often amusing especially where based on the average flow of the stream. In the well known Whitehall pond case which has just been tried before commissioners I presented in behalf of the city of Boston the following table and diagram which I think shows the effect of a diversion of a stated quantity of water daily from a stream. Whitehall pond is a tributary of the Sudbury river and by my estimate it is capable of yielding 483,117 cubic feet daily.

The question was what effect this diversion would have upon a mill situated some distance down stream, in fact a number of miles below the junction with the main stream. The table exhibits so clearly the process of reasoning that it will be unnecessary for me to go into the details. I think for the first time the *necessary* waste, in the case of a development of power to the 4th month and the waste from lack of power to use the water, are shown. It will be evident to any one accustomed to the usual methods of developing water power that a reduction could very well have been made for water wasted in various ways at the wheels, but in this particular case it was not thought wise to make this subtraction, it being the intention to give a liberal estimate of the amount of water that could be used or *available power*. The diagram shows what an enormous amount of waste occurs even with a mill pond of considerable capacity and where a very high development of wheel power is assumed.

STAND PIPES AND THEIR DESIGN.

BY

FREEMAN C. COFFIN, C. E., BOSTON, MASS.

There has been very little written upon the subject of stand pipes, in consideration of their importance in water works construction and their great and growing use.

The literature of the subject is extremely scanty. I do not know of any book upon it nor have I been able to find any treatment of it in any general work on water works construction.

There are a few good papers and some writing of a fragmentary nature.

Mr. John F. Ward has an article in the Engineering News of July 24, 1886, which contains many practical ideas in relation to construction.

Mr. A. H. Howland read a paper before the Engineers' club of Philadelphia, in 1887, in which he made some valuable suggestions and gave a table of minimum thickness of top plates and other dimensions.

Mr. B. F. Stephens read a paper before the American Water Works Association which is published nearly in full in the Engineering News of October 6, 1888. This paper contains a discussion of several failures and also a valuable statistical table of many stand pipes in use. The table was published in the Engineering News of October 13, 1888.

The Engineering Record in the issues of April 25 and May 2, 1891, published in full a paper read by Mr. W. Kiersted before the Rensaelaer Society of Civil Engineers some time before. In this paper is a theoretical discussion of the strains upon a stand pipe with several formulæ and a useful table of rivets and spacing. I shall refer to these papers in my discussion of the subject.

There are one or two other papers of which I have seen the name, but had not the opportunity to consult them.

The first question in the design of a stand pipe is the size, the element of height coming first, and this is largely determined by natural conditions. It is not within the province of this paper to discuss the head required to provide suitable fire streams, for that question is decided quite independently of the stand pipe, and when that is fixed in any particular case, the topography of the locality will determine the height or length of the stand pipe.

In a hilly country it must be a compromise between a heavy pressure in the low portions and a light pressure on the hills.

In this section an elevation can usually be found where it need not be of excessive height in order to give a satisfactory pressure over the greater part of the territory to be served; but different conditions prevail on the level lands of the west, and the influence of these is seen in the types of stand pipes built in the different sections. In the eastern states the present tendency is toward a stand pipe, or more correctly, a tank of large diameter, whereas on flatter country they are nearer what the name implies, a *stand pipe* or water tower, with great height and small diameter, the result of the ne-

cessity of obtaining sufficient head, combined with the desire for small expenditure.

I believe that the height, as it is the first element to determine upon, has also been generally considered the most important, and the holding capacity has nearly always been rated as the area of cross-section multiplied by the entire length or height, with the result of building a tall pipe of small diameter and vice versa. A desire for saving in first cost also tends in the same direction.

But where the subject is examined rationally it will be seen, I think, that the question of diameter is almost entirely independent of that of height. The efficient capacity must be measured by the length from the high water line to a point below which it is undesirable to draw the water on account of loss of pressure for fire supply, whether that point is the actual bottom of the stand pipe or above it. This allowable fluctuation, I should say, ought not to exceed 50 feet in most cases in a well designed system. This makes the diameter dependent upon two items, the first of which is the amount of the consumption during the ordinary interval between the stopping and starting of the pumps. This should never draw the water below a point that will give a good fire stream and leave a margin for still further draft for fires.

The second item is the maximum number of fire streams and their size, which it is considered necessary to provide for, and the maximum length of time in which they are liable to have to run, before the pumps can be relied upon to reinforce them.

Diagram No. 2 gives the number of feet which water in tanks of different diameters will be lowered by different populations using 60 gallons per head per day with the pumps running 8 and 10 hours in the day time, and being stopped at night.

This raises the question of the relative draft at different times of the day, and it would be of great value if data could be collected and examined to show the ratio of maximum and minimum draft per minute to the average draft per minute for the entire day. The last is easily found from the pumping records, but the writer recently having need to know approximately the maximum draft, could find no figures relating to it.

Through the kindness of Mr. Byron I. Cook, superintendent of the Woonsocket Water Works, where there is a recording pressure gauge on the stand pipe, I obtained the cards for one month, and computed the maximum hourly flow for each day. This case as you may see on diagram No. 5, plotted from one day's record, gives the hourly maximum 86 per cent. greater than the daily average. The average draft per minute from 6 p. m. to 7 a. m. was but 46 per cent. of the average daily draft per minute, and this was used as the basis of calculation for diagram No. 2. This was but one day in one city, but you may see that the daily average is close to the monthly average by reference to the diagram.

Diagram No. 3 shows to what depth any number of fire streams of 200 or 250 gallons per minute will draw the water in tanks of different diameters in one hour, which should be the maximum time before the pumps can reinforce the tanks.

There is another reason for making the diameter large, and that is to provide for stability against wind pressure when empty. The writer has seen a tank 30 feet in diameter and 113 feet high lifted ¾ of an inch from its foundation on one side, by the force of a strong wind, but not one of the strongest winds we have in this section.

The following table gives the height of stand pipes beyond which they are not safe against wind pressures of 40 and 50 lbs. per square foot. Area of surface taken is the height × ½ the diameter on account of the cylindrical form.

Table of heights of stand pipes that will resist wind pressure when empty by their weight alone.

Wind of 40 lbs. per sq. ft.		Wind of 50 lbs. per sq. ft.
20 feet diameter,	45 feet high.	35 feet high.
25 " "	70 " "	55 " "
30 " "	150 " "	80 " "
35 " "		160 " "

To have the above degree of stability, the stand pipes must be designed with the outside angle iron at the bottom connection as shown in Fig. 1, plate 6.

I consider any form of anchorage that depends upon connections with the side plates near the bottom, as unsafe. If attached at points on the side plates, as all forms of brackets and holding down bolts are, a strain comes upon the plate that it is not usually designed to bear, in addition to the strain caused by the water pressure, although it may be said that both strains will not come together. But aside from that, the strain which the wind pressure brings upon the horizontal joints is not lessened by this method which holds the bottom to the foundation, and leaves the rest of the pipe to resist the wind as a cantilever, whereas by suitable guys the wind pressure is resisted by tension in the guys, and the stand pipe is relieved from wind strains that tend to overthrow it. The guys should be attached to a band of angle or other shaped iron that completely encircles the tank, and rests upon some sort of bracket or projection, and not be riveted to the tank. They should be anchored at a distance from the base equal to the height of point at which they are attached, if possible.

The best plan is to build the stand pipe of such diameter that it will resist the wind of its own stability.

The foregoing suggests that a tank might be elevated upon a foundation or base, as the top 50 feet is all that will give the proper efficiency as storage capacity, and in fact, this has been done in some cases, and this part of the subject will be touched upon in the latter part of this paper.

The height and diameter having been decided upon, the material of which to build is the next point to be considered. Stand pipes are built successfully both of iron and steel, and I suppose the final word cannot yet be said in regard to the comparative merits of the two, everything being considered, but it may be a suggestive fact that all or nearly all failures have been of steel, unless I am misinformed. This may be due only to poor design or

construction, which it certainly was in some cases. This brings up the subject of failures and their causes, and I will mention a few of the noteworthy ones.

There have been a number of remarkable failures of stand pipes in the United States within the past ten years.

Defiance. O. This was one of the later failures, and occurred March 29, 1891, and was fully described in the Engineering Record of April 11, 1891. This pipe was 22 feet in diameter and 140 feet high, built of steel, with the lower ring $\frac{5}{8}$ inches thick; strength of material not stated, but to have a factor of 3 at the joints, it should be 65.000 lbs. ultimate T. S Upon water being first let into the tank, a leak was discovered, and it was found to be from a crack between the vertical rows of rivets in a plate in the second ring from the bottom. The sediment in the water soon stopped the leak, and nothing was done about it. On the morning of February 8, the gauge showed 60 feet of water in the tank or 80 feet below top, and upon starting the pumps, the engineer heard a loud report and saw ice thrown 10 feet above top of stand pipe, and it rocking violently Upon investigation, another crack was found in the same sheet, near the opposite end, from a rivet hole, and about six inches long. The leak stopped as before, and the pipe was used until March 29, when it burst, wrecking itself, and the engine house near by. Should say the cause was a poor sheet, combined with gross carelessness in its continued use.

Senaca Falls. N. Y. This pipe, which burst in 1887, was 130 feet high and 30 feet in diameter, of steel plates $\frac{5}{8}$ inches thick in lower ring, with 50,000 T. S. The lower ring should have been 1 inch thick to have a factor of 3 at the joints. The steel was said to be a coarse, poor grade, as indicated by the fractures. The foundation of this pipe, which weighed full about 3000 tons or $4\frac{1}{4}$ tons for each square foot of the bottom, was said to be of the poorest description.

I think there is no doubt that this pipe was built of too thin plates. The Engineering Record gives the strain on the plates between rivet holes as 23,000 lbs. per square inch and on the rivets as 25,000 lbs. per square inch, which I find is the case figured from the dimensions given. This is .5 of the ultimate strength of the plate and .6 of the shearing strength of best rivets, leaving a small margin against chances of defects in work or material.

On the same day as the above, a stand pipe in Franklin, Mass., fell from its pedestal. This tank was 40 inches in diameter, and 35 inches high, with $\frac{5}{8}$ plates at the lower ring. The plate seems to be of sufficient thickness, and the failure was in the pedestal or foundation, caused by filling the tank before the masonry was dry. The base was of brick masonry 45 feet high and 35 feet in diameter outside, of two rings of brickwork, the outer 16 inches thick, and the inner 12 inches, and 12 feet in diameter. The bottom of the tank was supported by iron I beams. The pressure per square foot on the brickwork with the tank full would have been 7.8 tons. With the amount of water in it when it fell, it was about 5.6 tons per square foot. A detailed

description of these two failures may be found in the Engineering Record, November 5-12, 1887.

Gravesend, N. Y. Perhaps the most remarkable of all failures was this one in October, 1886. Certainly it was a remarkable stand pipe. Its total height was 250 feet, diameter for 70 feet of the lower portion 16 feet, the frustrum of a cone from 70 feet to 95 feet from the base, where its diameter was 8 feet, which size it held to the top, 155 feet more. The first five feet were of $\frac{7}{8}$ inch plates; the next 30 feet of $\frac{3}{4}$ inch and growing thinner to the top where it was $\frac{1}{4}$ inch. The thickness seems ample, if the metal were good. The Engineering News stated that the appearance of some of the bottom plates indicated poor and brittle metal. I think Mr. John F. Ward has attributed the failure to the right cause, which was that the top was drawn in, creating a tendency in the pressure of the water on the under side of the conical portion to lift the sides from the foundation, which it is reported to have done at first filling, when there were two rings of braces, of 24 in each ring, put inside and riveted or bolted to the sides and bottom, to prevent the sides from rising, or in other words, the bottom from bulging outward. Mr. Ward's letter may be found in the Engineering News of November 13, 1886. His figures are total pressure on the bottom of 1448 tons; total weight of water, stand pipe and fixtures, 855 tons, leaving a lifting force of 593 tons. I figure it a little differently. The total lifting strain equals the difference in area of the two diameters 8 and 16 feet or 151 square feet $\times$ the average pressure on the under side of cone or 5.23 tons per square foot equal 790 tons total pressure. To oppose this lifting force, there is the weight of the water in the annular space under the cone, plus the weight of the sides of stand pipe, about 440 tons in all, leaving 350 tons to be carried by the braces to prevent the bottom from bulging. The total strain will come at first on the inner ring or long braces, 24 in number, making 14.6 tons on each brace. Their section was $2\frac{1}{2}$ inches $\times$ 1 inch with probably a loss of at least 1 inch for rivet and bolt holes, leaving $1\frac{1}{2}$ square inches effective section, or a strain of nearly 10 tons per square inch or 20,000 lbs. The braces being an after thought, may have been ordinary rolled bars of a low tensile strength, and it is more than likely that they were attached in such a manner as to bring the strain that two should have carried upon one in some cases, which giving away, as Mr. Ward says, brought a shock upon the plate to which it was fastened, that was too much for its strength and rupture followed.

The whole design of the stand pipe was most absurd. To save weight probably it was made smaller at the top where the plates were light, and left large at the bottom, where the plates should be as heavy as if the full diameter were carried to the top; saving at the spigot and wasting at the bung, and the storage capacity reduced at the top where needed, and maintained at the bottom where it was not available. In fact the stand pipe was upside down. It was this that suggested the form of structure shown on plate No. 7 which is this one inverted, with the proportions changed. This failure was described in the Engineering News and Engineering Record.

I will mention one stand pipe that was overthrown by the wind, that at Kankakee, Illinois. It was 20 feet in diameter, and 124 feet high; first ring 11-16 inches thick, which was ample, with good metal. The top plates were too thin, being but ⅛ inch and were, in my opinion, the cause of the accident. The tank was anchored by six 1¼ inch bolts, fastened to the second ring of plates, and passing down into the foundation. It was empty at the time; velocity of wind estimated at 60 miles per hour.

The weight of the tank multiplied by its leverage, ½ diameter, was less than the wind pressure by *its* leverage, ½ the height, and the excess came upon the rods, straining them, according to my figures, from 5 to 10 tons per square inch, with wind pressure of 20 to 30 lbs. per square foot. This was in a normal condition, but when the top collapsed and formed a pocket, at once doubling the effect of the wind on that portion, and throwing the increase all upon the rods they parted, and the pipe fell.

But to return to our subject. There seem to be reasons for the use of steel in boilers, that do not exist in stand pipes. As I understand, it is desirable to keep the plates in boilers as thin as possible to prevent burning, and this the higher tensile strength of steel accomplishes. Steel is also said to be less liable to blister than iron, on account of the different structure of the two. I have been able to find no discussion on the comparative merits of the two materials in the construction of stand pipes, but of the above reasons for using steel for boilers, neither apply to stand pipes, and in fact, the extra thickness required in iron owing to its lower tensile strength, is an advantage, for as far as I know, iron rusts no faster than steel, nor thick plates faster than thin, and the loss is a smaller percentage of both thickness and strength in thick plates than thin. But the question of cost rules all things, and as I am told good steel can be furnished at least 25 per cent. less cost than good iron, if it can be shown that it is a safe material to build with, as probably a mild steel is, if properly proportioned, the majority of pipes will be built of that material.

The next element is the thickness of the side plates. There are many formulæ, more or less complex but it seems to me a simple question. The pressure on the sides is outward, and due alone to the weight of the water or pressure per square inch (I refer now to strain tending to rupture the plates vertically) and increases in direct ratio to the height and also to the diameter. The strain upon a section 1 inch in height at any point, is the total strain at that point, divided by two, for each side is supposed to bear the strain equally. The total pressure at any point is equal to the diameter in inches, multiplied by the pressure per square inch due to the height at that point.

It may be expressed in a formula as follows:

Let H=height in feet
" f=factor of safety.
" d=diam. in inches
" p=pressure in lbs. per square in.
" .434=p for 1 foot in height.
" s=tensile strength of material per square inch.
" T=thickness of plate

*then total strength on each side per vertical inch

$$= \frac{.434Hd}{2}$$

$$\text{or} = \frac{pd}{2}$$

$$T = \frac{.434Hdf}{2s} \qquad \text{or} = \frac{pdf}{2s}$$

There is one element in the above formula to be decided and one of the most important, i. e. what value to give f. This is of course a matter of judgment, but there is one point to be settled before we can use our judgment intelligently. It seems to be the general custom in boiler construction to define the factor of safety in terms of the tensile strength of the plate, as a factor of 5 for 50,000 lbs. iron, would give 10,000 lbs. as the safe strength but a moment's reflection will show us that as the joints are the weakest point in the construction, it is to them that we must look for the safe strength of the work, and we must first find the strength of these, which is dependent upon the strength of the plate, and when found may be expressed as a percentage of the strength of the whole plate, and then we may denote our factor in terms of the tensile strength of the plate, if more convenient, for we shall then know what we mean by it, and shall not be deceived by a high nominal factor of safety.

Some experiments with specimens of riveted joints show as high as 60 per cent. of the strength of the plate for single riveting and 75 for double. Fairbairn gives respectively 56 and 70 per cent. including friction in the joint. Of course the experiments with riveted specimens new and made for the purpose include the friction, but I do not think it is safe to rely upon it at all in stand pipes. When first built, and empty, they are not quite round; generally when filled, the water forces them into circular shape, and some of the joints must start a little. It is a fact that they do, and cause leaks; this would destroy the friction in these joints. Again, the joints are liable to rust between the plates where they cannot be painted; this would destroy the friction. Therefore it seems safe to disregard it, and compute the strength of the joint from the section of plate between the holes, and the shearing strength of the rivets. The crippling strength of the rivets need not be considered, as it is higher than the shearing strength in all dimensions of rivets and plates used in stand pipes. The aim must be to make the strength of the untouched plate equal to the shearing strength of the rivets.

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*Note error in formula in diagram No. 1.

I have calculated the joints on this basis in plates from ¼ to 1 inch in thickness, with different sizes of rivets and spacing of the holes, and as a result have designed a table of dimensions, which gives the highest results I could obtain. In these calculations I took the safe strength of the plate as 10,000 lbs. and strength of rivets in single shear as 7,500 lbs. The plates not only lose the metal in the holes, but the remaining metal is weakened by punching.

Prof. C. H. Benjamin, of Case Scientific school, recently described some tests of steel before the Civil Engineering club of Cleveland, where he found this loss to be 7.5 per cent. for punching over drilling as a mean of all his tests. He found that the loss due to the use of a spiral punch was only 3 per cent.

I find in a book on boilers by W. M. Barr, a series of tests of steel by Kirkaldy, in which the deterioration is 9.5 per cent.

In the same book are some tests of iron by Hooper and Townsend. In these the punched bars showed greater strength than drilled ones. They claim that the result was due to the use of a punch that accurately fitted the die, instead of having the die larger, leaving a conical hole, as is usual, their theory being that the plate is subjected only to direct vertical pressure, with no tendency to lateral or bursting strains, with punch and die the same size.

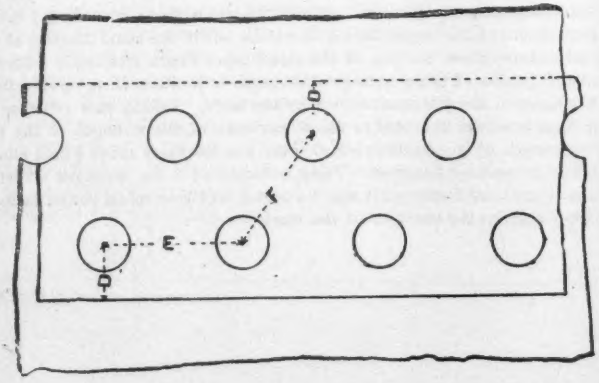
Mr. Barr also claims superiority for the spiral or shearing punch, with tests of bars punched with both spiral and flat punches, one hole of each in each bar, the bar broke through the hole of the flat punch each time.

In my calculations I assume a loss of 8 per cent. beside the loss of metal. In the right hand column are the percentages of strength of the joints in relation to the strength of the plate, which range from 59.5 per cent. in 1 inch plates to 65.5 in 5-16, for double riveted joints, and 70 per cent. for triple riveted joints in 1 inch plate. It would be better to use triple riveting than to exceed 1 inch plate.

The dotted line C. D. on diagram No. 1 shows to what height stand pipes of different dimensions may be built of 50,000 metal without exceeding 1 inch if triple riveted, and the dotted line A. B. on the left of the same diagram shows at what distance from the top of the stand pipes single riveting is sufficient in vertical joints. The percentage of strength in joints is in inverse ratio to the thickness of the plates as shown by the table. Taking this table as the basis, I assume that it is safe to use 60 per cent. of the strength of the iron as the strength of any double riveted joint, but all plates above ¾ inch should be drilled instead of punched. Using a factor of 5 for strength of whole plate, or a nominal factor, as it may be called, will give us an actual factor of 3 at the joints for the strength of the work.

TABLE OF RIVETS AND SPACING.

	Thickness of Plate	Rivets Diam.	Length	Center to Edge of Plate	Center to Center of Rivets	C to C of Rows Diagonally	Per Ct of Strength of Whole Plate	
	A	B	C	D	E	F		
Single Rivet	1/4	5/8	1 1/4	1	1 1/16		54.5	Punched
" "	5/16	11/16	1 1/2	1	1 11/16		52.5	"
Double "	5/16	5/8	1 1/2	1	2 1/4	2	66	"
" "	3/8	3/4	1 3/4	1 3/16	2 5/8	2 1/8	66	"
" "	7/16	13/16	2	1 1/4	2 3/4	2 1/4	64.7	"
" "	1/2	7/8	2 1/4	1 3/8	2 7/8	2 7/16	63	"
" "	9/16	15/16	2 1/2	1 1/2	2 15/16	2 9/16	63	"
" "	5/8	1	2 3/4	1 9/16	3	2 3/4	62	"
" "	11/16	1 1/16	3	1 5/8	3 1/8	2 7/8	61	"
" "	3/4	1 1/8	3 1/4	1 3/4	3 1/4	3	60	"
" "	13/16	1 1/8	3 1/8	1 3/4	2 7/8	2 7/16	61	Drilled
" "	7/8	1 1/8	3 1/2	1 3/4	2 7/8	2 7/16	59.3	"
" "	15/16	1 9/16	3 3/4	1 7/8	3	2 3/4	59	"
" "	1	1 1/4	4	1 15/16	3 1/8	2 7/8	59	"
Triple "	1	1 1/8	3 1/4	1 3/4	4	3 1/2	70	"



Mr. Howland in his paper, places the strength of the joint at 75 per cent. in double, and 66 per cent. in single riveting, based upon government test. I presume this includes friction. He however, advocates a factor of 4, which would make the joints equal to a factor of 3.2 on basis of 60 per cent.

The vertical joints require more care in design and workmanship than any other portion of the structure, and the *rivet iron should be of the best quality.* The strain upon the horizontal joints is caused by the weight of metal above the joint, and is small in proportion to the strength of the joint, and by the pressure of the wind in its tendency to overturn the tank, which would only come upon the tank when it was empty, or partially so, and as I regard it, would not be added to the strain caused by weight of metal.

The amount of the wind strain per square inch of metal at any joint, can be found by the following formula, in which

H=height of stand pipe in feet.
T=thickness of plate in inches.
w=wind pressure per foot in height above joint.

$w = \frac{Dp}{2}$ where D=diameter in feet.

p=wind pressure per square foot.
m=average leverage or moment about neutral axis or central points in the circumference.
or m=sine of 45° or .707 times the radius in feet.

then strain per square inch of metal of plate $= \frac{H^2 w}{2 \text{ (circ. in ft.) } mT}$

I have not had time to work out a diagram of strains caused by wind pressure as I should like to have done, but with wind pressure of 40 lbs. per square foot all pipes built of thickness called for by diagram No. 1 up to 30 feet in diameter, are amply strong with single riveted horizontal seams if not higher than the limit given in the list of stability against wind pressures, and if properly secured by guys, will be all right for any height. A 30 foot stand pipe should be double riveted in the horizontal seams below 100 feet from the top. Pipes 35 feet in diameter and over, will be safe with single riveted horizontal joints, up to the limit of 1 inch bottom plates at least.

The only other strain to be provided for is the wind pressure tending to collapse the top. Mr. Kiersted gives in his paper a formula to find the thickness of plates to resist this, but says it is a crude and approximate method. I think it must be left to the judgment.

On the diagram for thickness where the full oblique lines change to dotted oblique and full vertical lines, is indicated the thickness of the top for different diameters. The top plates should never be less than these vertical lines indicate, and must be thoroughly reinforced by a heavy angle iron, well riv

eted to the plates at the top, and encircling the tank. The length of the full vertical lines, shows how far down from the top, this minimum thickness may be carried.

This diagram, which is on the left hand of the large diagram No. 1 gives total strain on a 1 inch vertical section of side plates for all diameters shown on oblique lines and all heights to 160 feet. The scale of strain is given at the bottom. It also shows the thickness required at any point with material of 50,000 lbs. T. S. with nominal factor of 5, equal actual factor of 3 at joints. The scale of thickness in sixteenths of an inch given at the top. I have made inquiries and calculations to find by what fraction of an inch it is most economical to change the thickness of side plate, and find that it pays to reduce by sixteenths, and it does not pay to make the reduction less than that. The thickness of plates may be read directly from the diagram, and the fraction of a sixteenth should be added rather than taken off when the thickness does not come on the line. The scale of height is divided every five feet, as that is the common width of plate, but other widths may be taken by estimation or by scale.

There is no strain upon the bottom plates when resting upon a solid foundation, and the thickness must be left to the judgment. It must be sufficient to caulk properly to resist the pressure of the water, and also to allow for corrosion on the under side. In computing the diagram of weight, I used bottom plates approximately 75 per cent. of the thickness of lower ring of side plates in each case.

I think there are no other elements in the general design. Matters of detail must be largely left to individual taste. The bottom connection with sides is important; Fig. No. 1, plate 6, shows the most desirable form; this brings the weight of the sides well in upon the foundation, and also gives greater stability against wind pressure.

The side rings are generally made larger at the top to allow the caulking to be done on the outside of the stand pipe and on the top edge of the plate in the horizontal seams. Sometimes the rings are the same size at top and bottom, and every alternate ring is inside of the one next above and below it. This is not quite as convenient in caulking, but the tank is more likely to be true and round, and has a neater appearance. Perhaps the most difficult work in erection is the beating down of the edges of the plate where three thicknesses meet. In thick plates, this is very difficult, and they have to be heated in place by a basket fire to do it, and when finished an unsightly spot is left at the joint. I think a better method would be to make the joints as shown in Fig. 5, plate 6. The rings alternate in and out, and only the horizontal joints are lapped; the vertical joints are made with butt straps as shown in the figure; this avoids beating down the plates as they are butted at the vertical joints. Although there would be a small increase in weight of metal and of riveting due to the butt straps, yet it would result in a material economy in a pipe with plates over $\frac{1}{2}$ inch thick, and improve its appearance. The caulking should be done on the outside except on the bottom where the outside cannot be reached.

In a cold climate there should be no bars, braces, pipes or ties inside the stand pipe. A light ladder seems to be a necessity, but it should be as plain as possible, and truly vertical.

Diagram No. 1 giving the cost and weight of stand pipes from 20 to 100 feet in diameter and up to 160 feet high was computed and plotted on the basis of design suggested in this paper. Ten per cent. was allowed on weight of all iron work for laps and rivets, and excess of metal over the standard thickness. This would not cover the loss from shearing down plates that were not rolled to the right size, but I believe they come from the mill cut to order. The changes in thickness was made by sixteenths of an inch, and the weight of each diameter carefully computed for every 10 feet in height, and includes the bottom and angle iron. The cost is figured at 5 cents per pound erected and painted inside and out, 3 coats. This will cover the cost built of best iron at the present time, of tanks whose total weight is not less than 50 tons; for weights less than this, the cost might be higher; but contract prices vary from time to time, and it is only claimed for the diagram that as close an estimate may be taken from it as can be obtained in any other way, and if the market warrants a different basis than 5 cents per lb., the result obtained from the diagram may be multiplied by the proper ratio. If a balcony is required, the table of cost at bottom of diagram gives the cost for different diameters of stand pipe of a plain balcony with hard pine floor. Ladders will cost approximately 50 or 60 cents per lineal foot, and spiral stairs may be built in a neat, plain style for about $5 for each step including hand railing. The weight and cost are reprosented on the diagram by curves from the upper left hand corner. The capacity in U. S. gallons is represented by oblique lines from the upper right hand corner. The scale of weight and cost is at the top of diagram, and of capacity at the bottom. Diagram No. 7 gives the cost of good cement rubble masonry foundations, with granite coping 12 inches deep all around. Three different scales give the cost at $4, $5 and $6 per cubic yard including earth excavation. These prices probably cover the extreme limits in different localities.

With underground water supplies, it is necessary to cover the stand pipe. A good form of roof is a conical one made of plates $\frac{1}{8}$ inch thick lapped and riveted, and attached by iron clips to the angle iron at the top of the stand pipe, also to a sufficient number of interior horizontal ribs of angle iron to give it resistance to the wind. Plate No. 7 shows the general appearance of this form of roof, and diagram No. 8 gives weight and cost of it if built without ornamentation. Two scales give the cost at 8 and 10 cents per lb., the latter price being probably the safer.

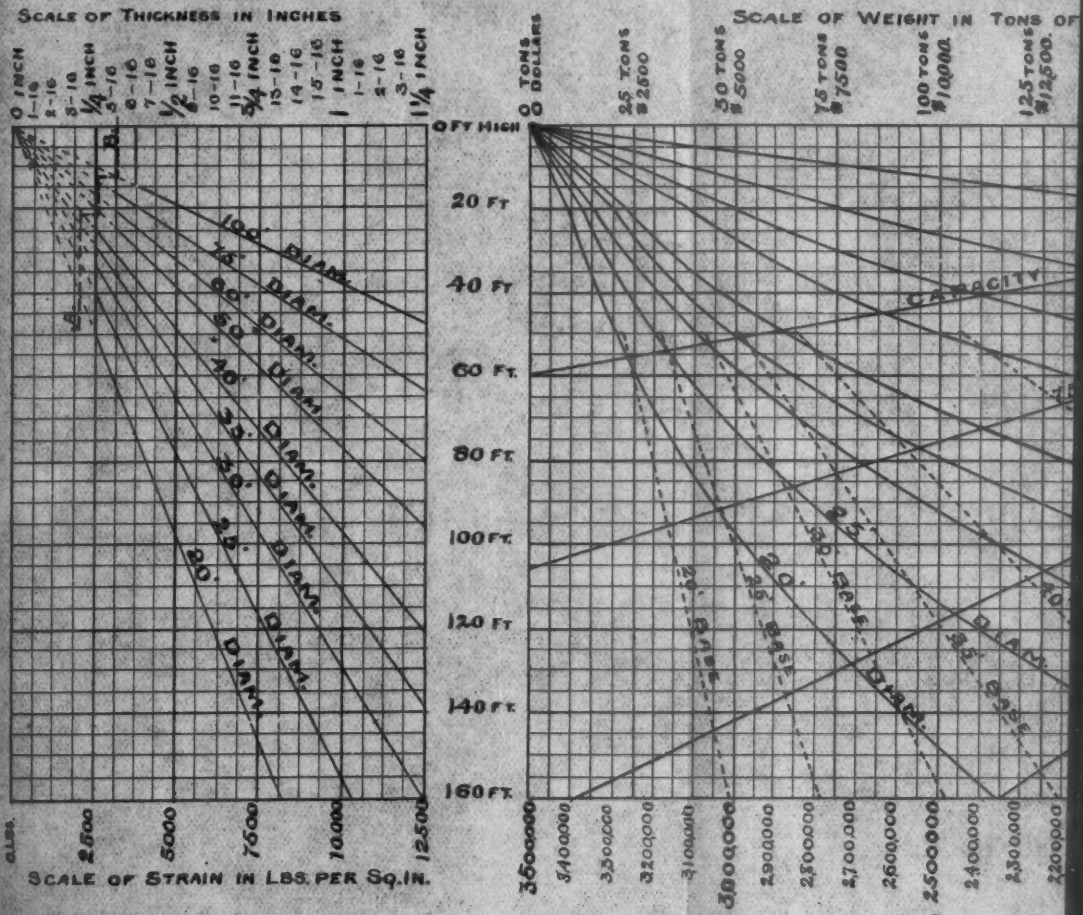
From the diagrams given with this paper, estimates of cost of stand pipes of the design and sizes given, with all parts of the same, may be taken in a few minutes, that will be as close as any that may be made with hours of computation or obtained in any way except by actual bids.

ELEVATED TANKS.

I should like to notice a number of elevated tanks that have been built, but this paper is already much longer than I intended it to be, and I will just speak of the Norton water tower of the Liverpool Water Works, and a form proposed by Johnson & Fladd, civil engineers. The Norton tower is an immense tank or basin of mild steel 82 feet diameter, with vertical sides about 10 feet high, with a bottom in the shape of an inverted dome, 19 feet deep. This is supported by a tower of stone masonry, the whole making a handsome structure about 125 feet high. A full description of this remarkable tower may be found in the Engineering Record of October 3, 1891.

The form of tank proposed by Messrs. Johnson & Fladd is illustrated and described in the Engineering News of August 15, 1891. This form of tank has an inverted dome shaped bottom, and is elevated upon a base of steel columns, and makes a pleasing design at reasonable cost.

Plate No. 7 shows an elevated tank in which no claim is made of originality in any of its parts, but the whole design is somewhat different from any I have seen, and seems well adapted to economical construction. The bottom is conical, which avoids the necessity for support, except at the point where it is connected with the sides, and is less expensive to construct than a spherical form. The plates where they connect with the sides should be the same thickness as the lower ring of side plates, and may be reduced in thickness toward the centres, where the supply pipe enters, through a stuffing box of composition, a detail of which is shown on plate 6, Fig. 4. The base is of steel, Z bar columns, of which a section is shown in Fig. 3. plate 6, well braced and counter braced to resist the wind pressure. The outside line of the columns is a parabolic curve, made up of sraight lengths of the columns. The drawing shows them in lengths of 20 feet, but they may be made shorter with a little more expense for bracing if necessary to make a smoother curve. This form of column is well adapted to this work, as the connections are so easily made, and the whole surface is exterior and accessible for painting. The connection with the tank is made to an extension of the side plates as shown in Fig. 6, plate 6, which shows the form of construction to resist the compressive strains in the sides caused by the water on the conical bottom. The cost of this form of base for different diameters of tanks 50 feet high, is represented by the dotted oblique lines marked "base" on diagram No. 1. Their divergence from the curved lines for the same diameters shows graphically the difference in cost of the base and a continuous stand pipe. The cost is figured at 5 cents per lb. for the base, but I am told by a firm that does a large amount of similar work, that all of the steel in the base can be erected and painted for 4 cents per lb.



Cost of Plain Balconies

20 Ft	Diameter	$165.00
25 "	"	205.00
30 "	"	250.00
35 "	"	285.00
40 "	"	320.00
50 "	"	410.00
60 "	"	500.00
75 "	"	615.00
100 "	"	820.00

Data

Tensile Strength of Material = 30,000 Lbs
Nominal Factor of Safety = 5
Actual " at Joints = 3
Cost of Material Erected $.05 Lb

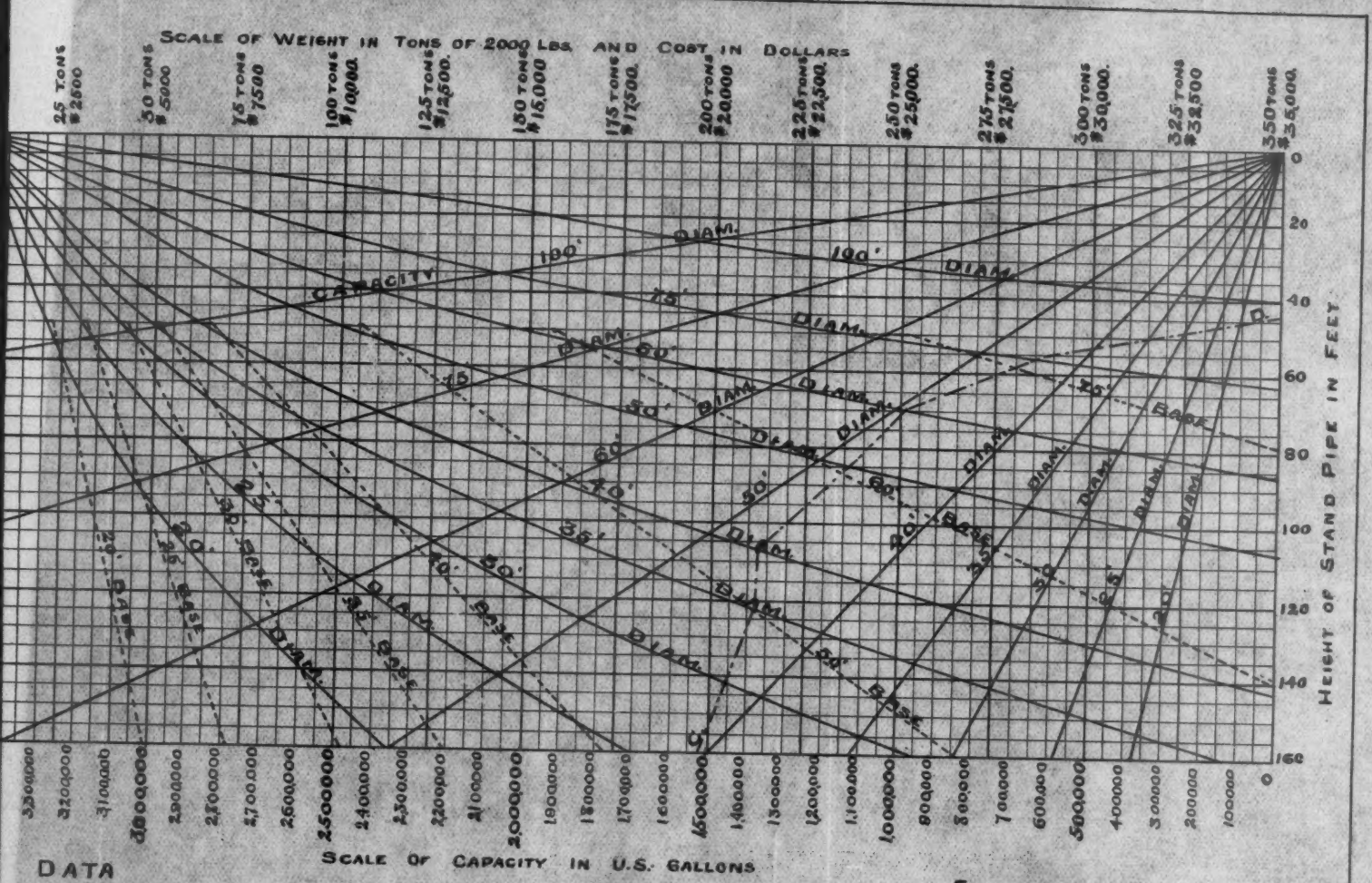
Strain Formula

$$S = \frac{(Hd).434}{2}$$

$$T = \frac{(Hd).434}{2(M \times factor)}$$

H = Height in Feet
d = Diam. " Inches
S = Strain in Lbs. per Ver
of Metal on each Side
M = T.S. of Material
T = Thickness of Plate in

D I A
Stand
Fra



DATA

STRENGTH OF MATERIAL = 50,000 LBS
L FACTOR OF SAFETY = 5
" AT JOINTS = 3
MATERIAL ERECTED = .05 LB

STRAIN FORMULA

$$\frac{(Hd)434}{2}$$

$$\frac{(Hd)434}{2(M \times \text{factor})}$$

H = HEIGHT IN FEET
d = DIAM. " INCHES
S = STRAIN IN LBS. PER VERT. INCH OF METAL ON EACH SIDE
M = T.S. OF MATERIAL
T = THICKNESS OF PLATE IN INCHES

DIAGRAM No. 1

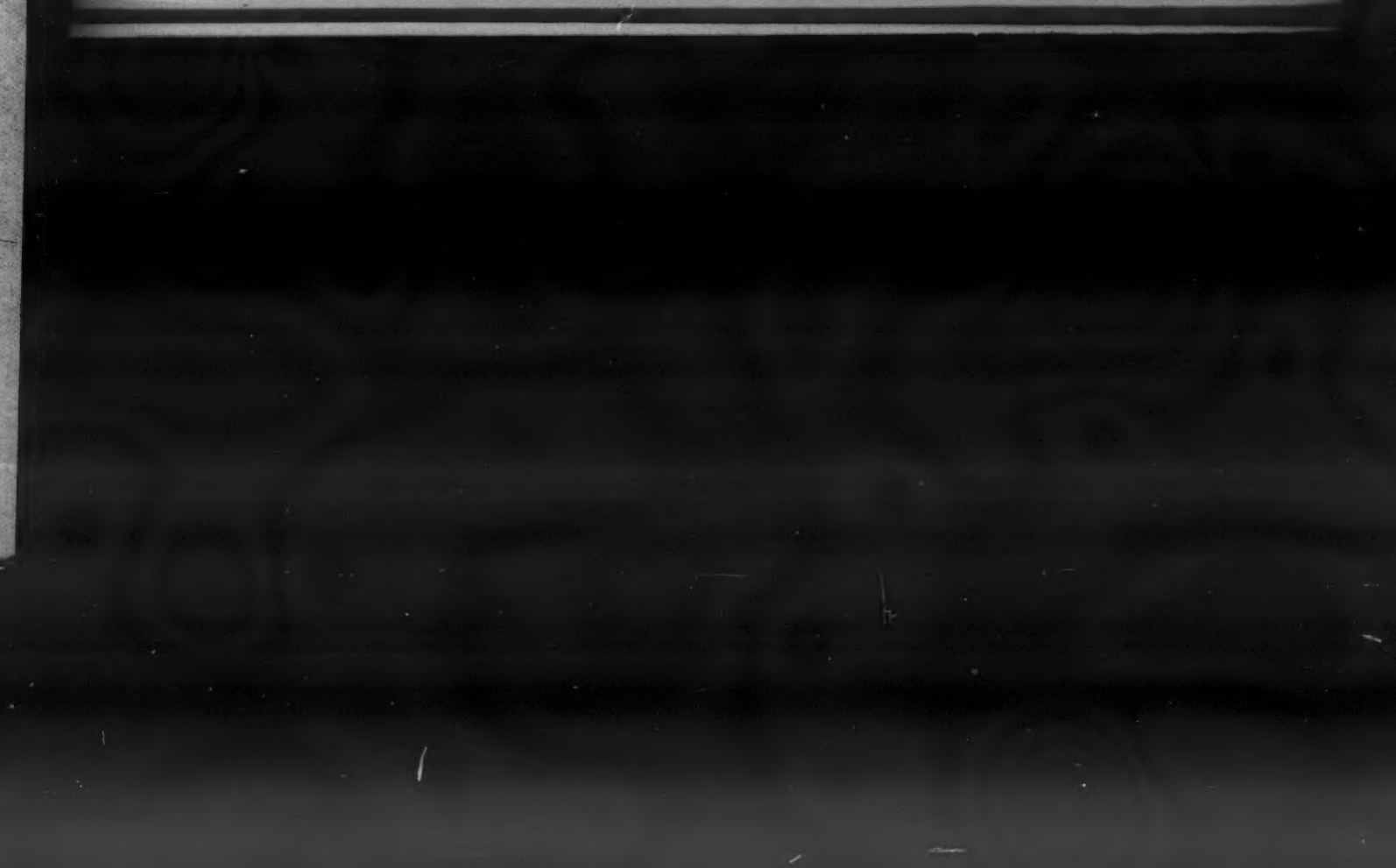
WITH PAPER ON
STAND PIPES AND THEIR DESIGN
BY
FREEMAN C. COFFIN

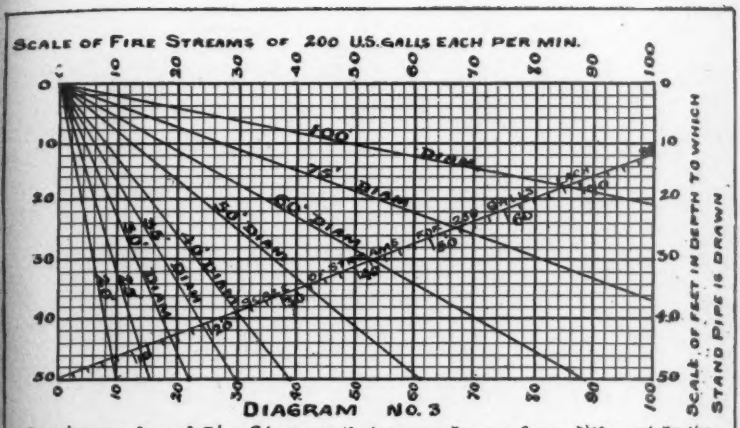
EXPLANATION

FULL CURVED LINES FROM UPPER LEFT HAND REPRESENT WEIGHT AND COST OF SIDES AND BOTTOM

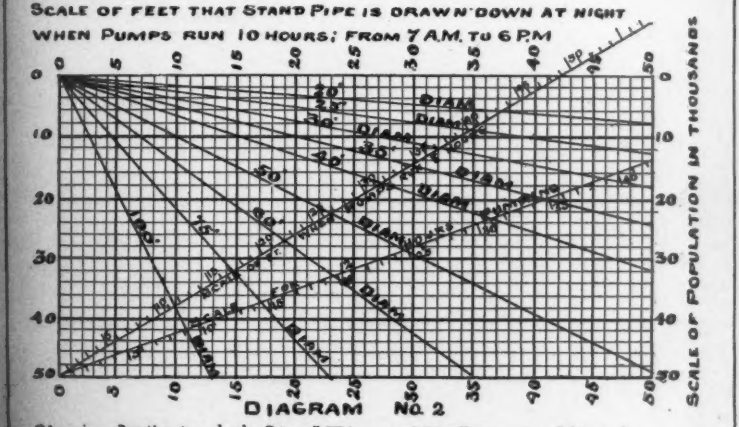
DOTTED STRAIGHT LINES REPRESENT WEIGHT AND COST OF FRAMED STEEL BASE

FULL STRAIGHT LINES FROM UPPER RIGHT HAND CORNER REPRESENT CAPACITY IN U.S. GALLONS





Showing number of Fire Streams that can be drawn from different depths of Stand Pipes of various diameters per hour


SCALE OF FEET THAT STAND PIPE IS DRAWN DOWN AT NIGHT WHEN PUMPS RUN 10 HOURS; FROM 7 A.M. TO 6 P.M
SCALE OF POPULATION IN THOUSANDS
DIAGRAM NO. 2

Showing depths to which Stand Pipes will be drawn by different populations in different intervals of time between pumping

With paper on STAND PIPES AND THEIR DESIGN BY FREEMAN C COFFIN

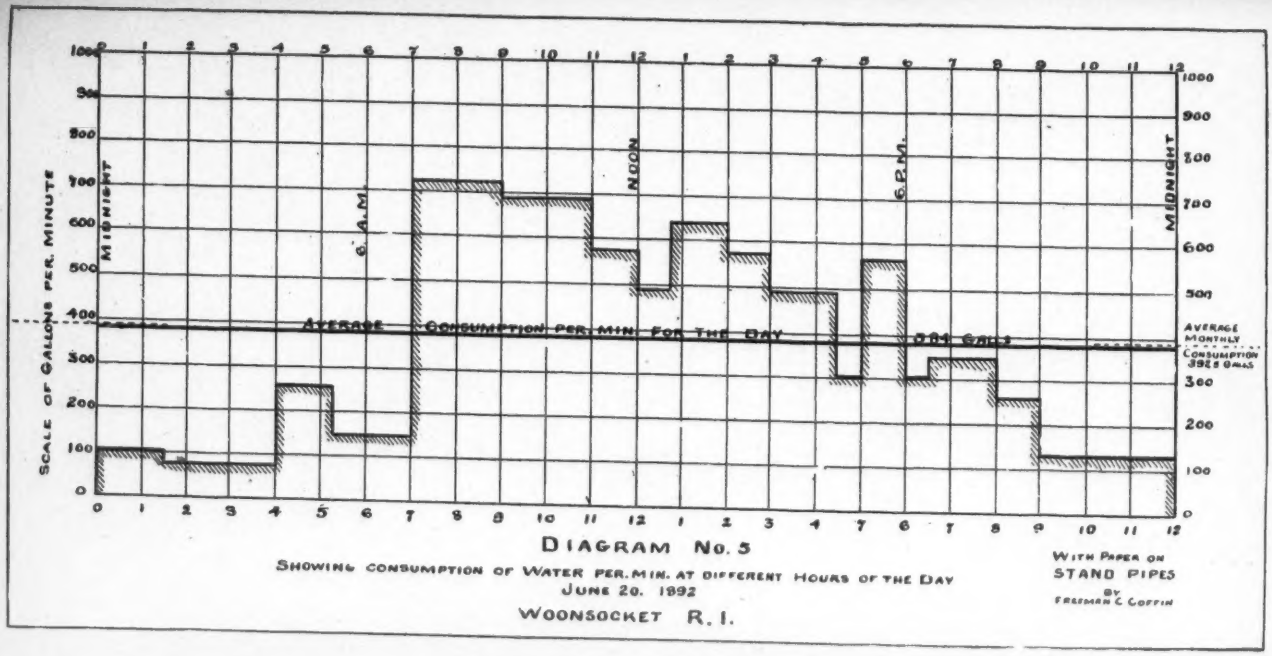
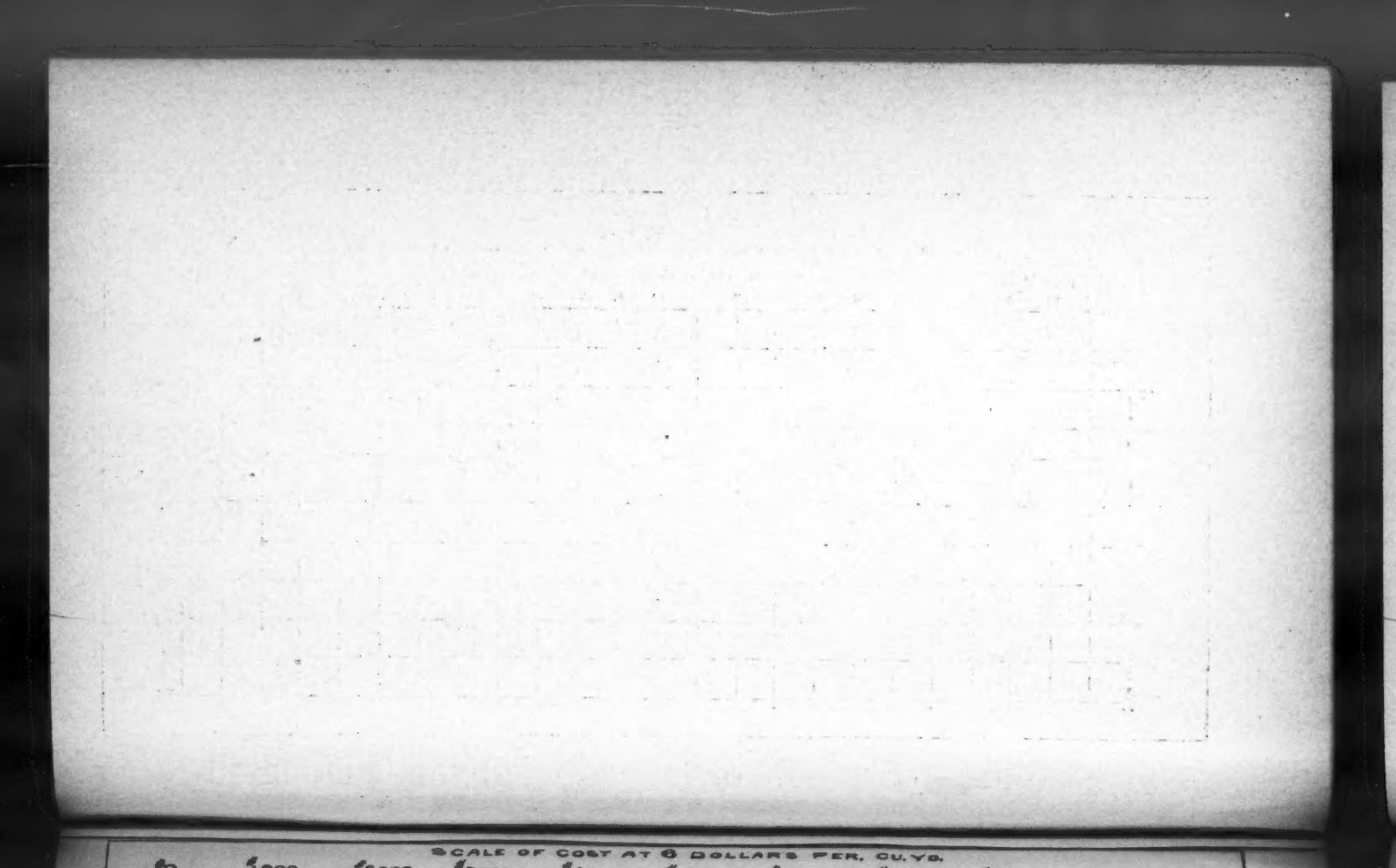


DIAGRAM No. 5

SHOWING CONSUMPTION OF WATER PER. MIN. AT DIFFERENT HOURS OF THE DAY
JUNE 20. 1892

WOONSOCKET R. I.

WITH PAPER ON
STAND PIPES
BY
FREEMAN C. COFFIN


SCALE OF COST AT 6 DOLLARS PER. CU.YD.

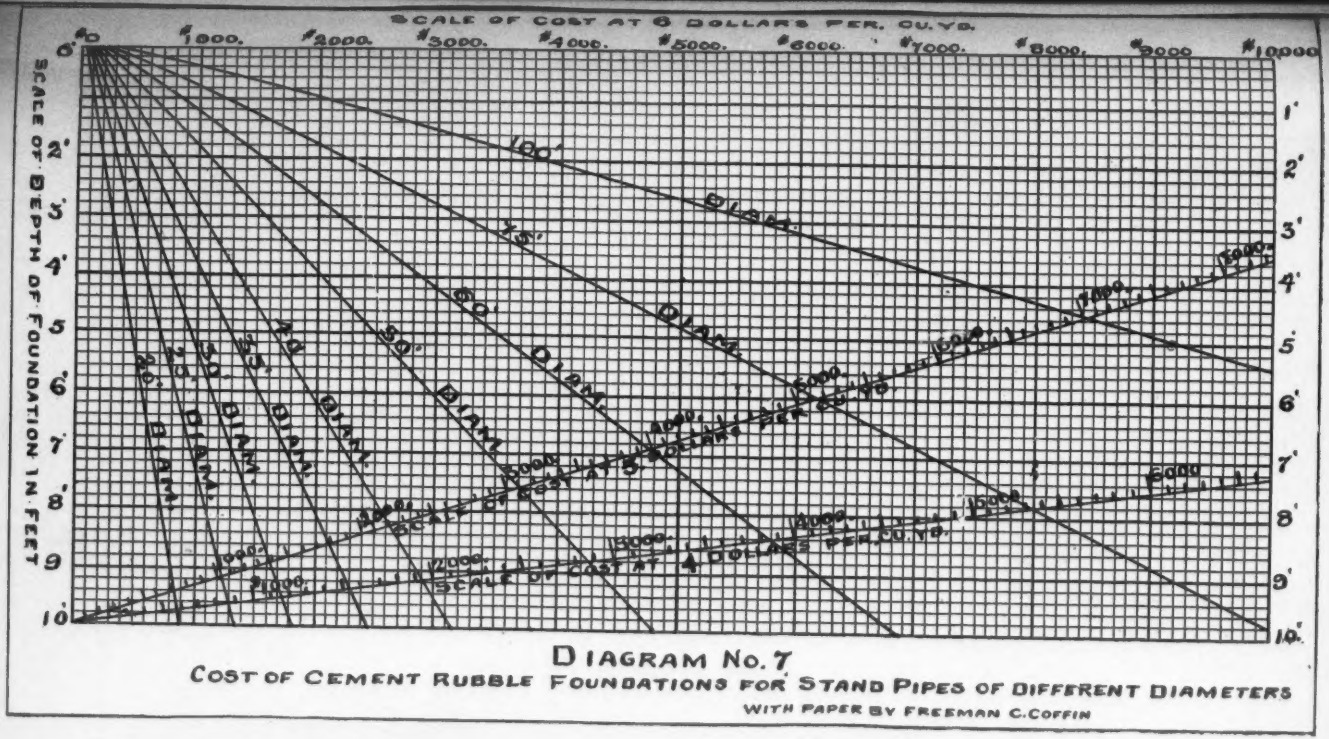


DIAGRAM No. 7

COST OF CEMENT RUBBLE FOUNDATIONS FOR STAND PIPES OF DIFFERENT DIAMETERS

WITH PAPER BY FREEMAN C. COFFIN

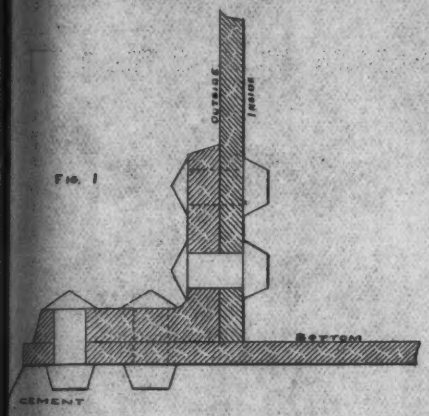


FIG. 1

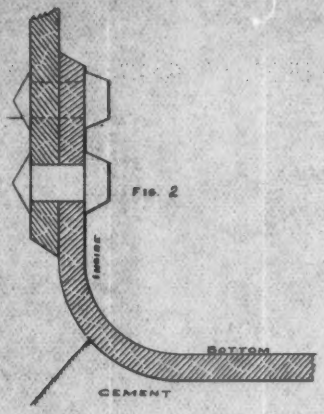


FIG. 2

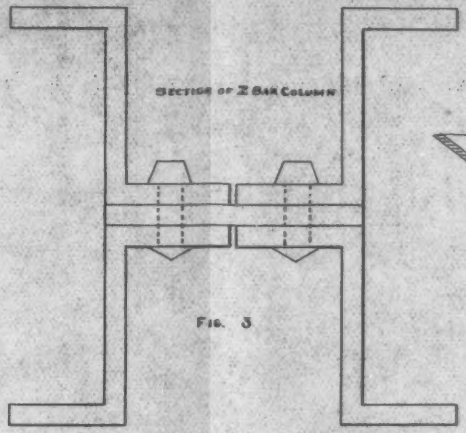
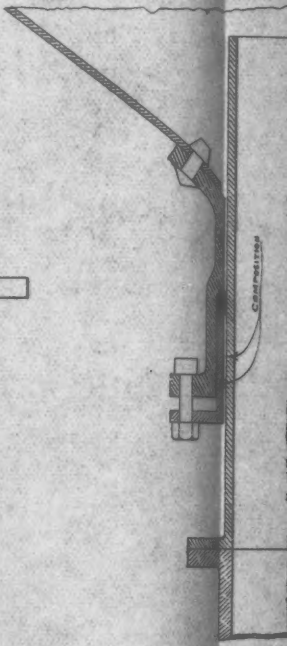


FIG. 3



WITH P

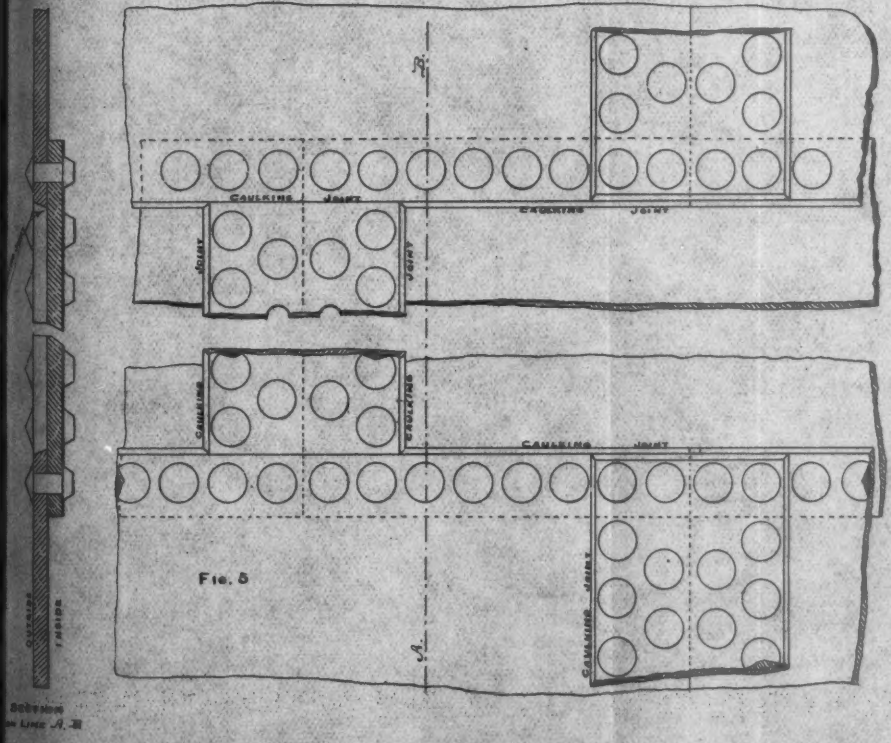


FIG. 5

SECTIONS ON LINE A-B

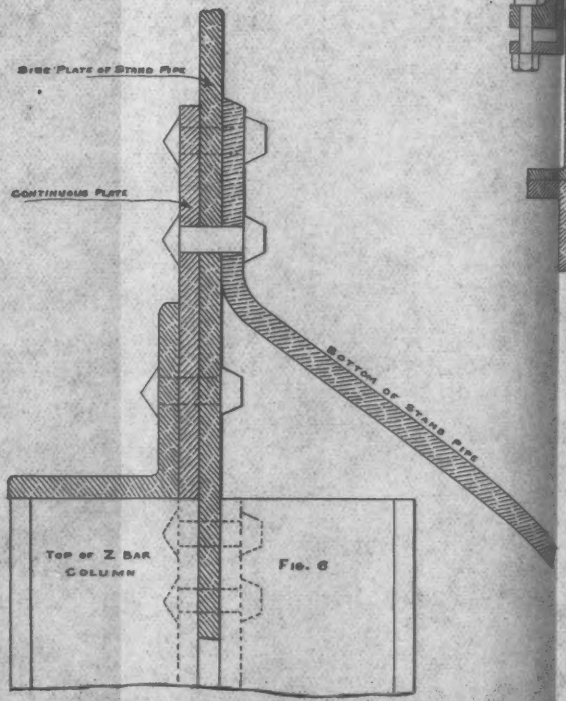


FIG. 6

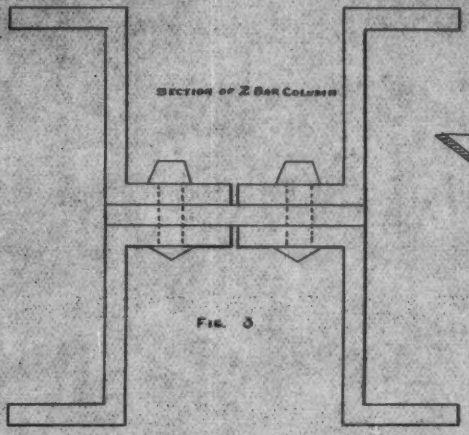


Fig. 3

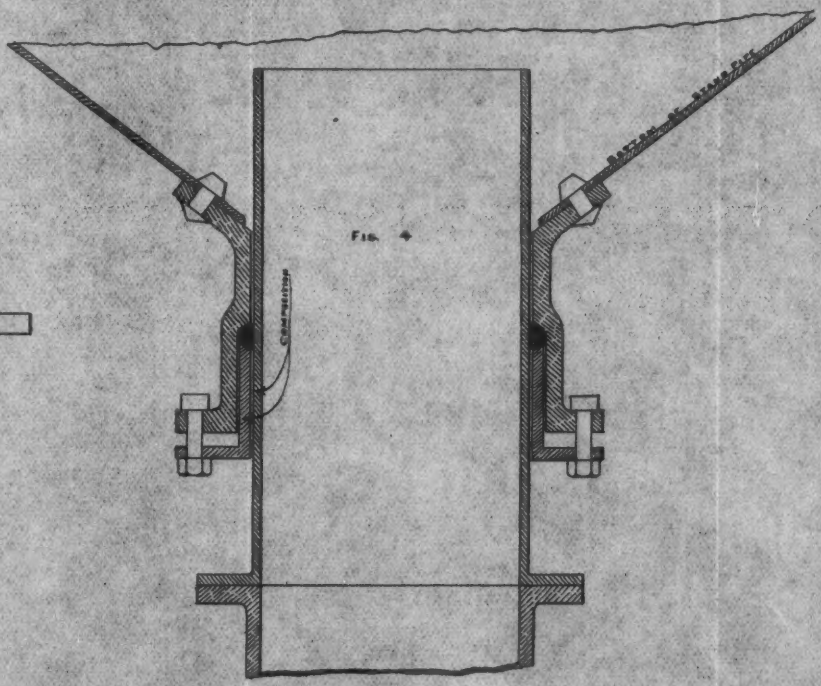


Fig. 4

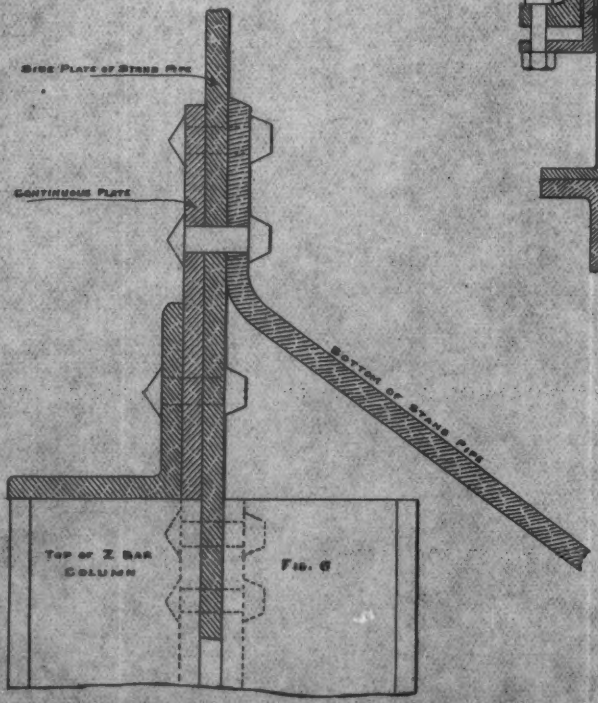
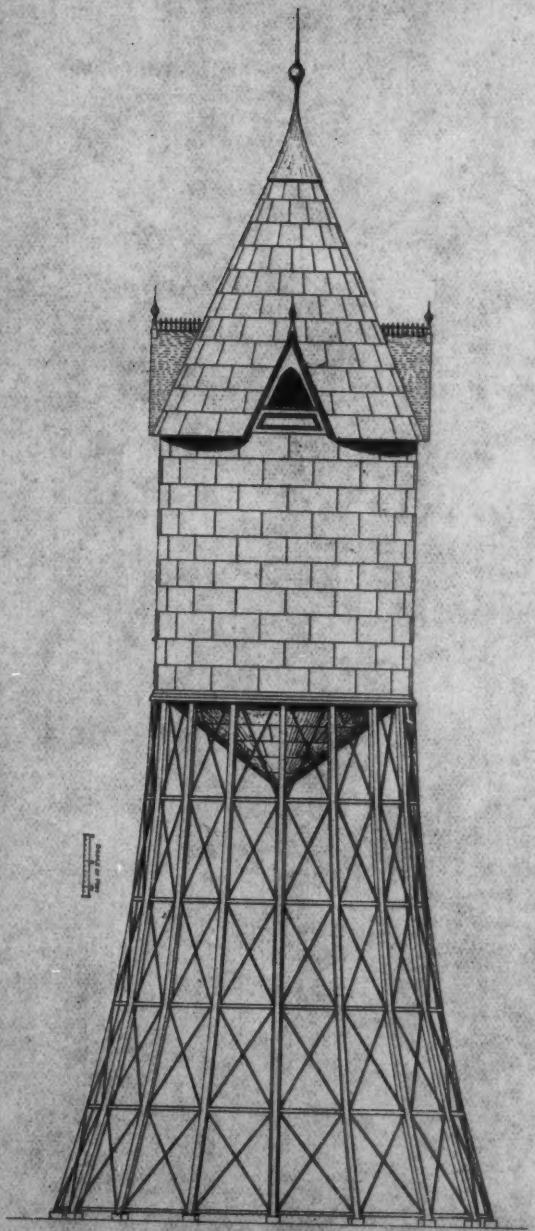


Fig. 6

Plate No. 6

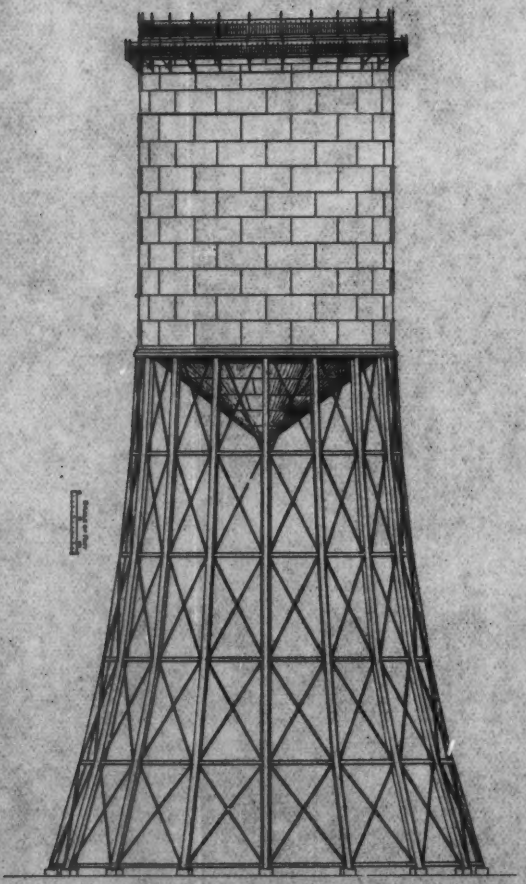
With Paper on STAND PIPES and THEIR DESIGN

By Freeman C. Coffin



ELEVATED TANK 50' DIAM. 30' HIGH. FRAMED STEEL BASE 100' HIGH
WITH PIPES AND STAND PIPES AND THEIR DETAILS BY FOREMAN C. O'BRIEN

PLATE No. 7



ELEVATED TANK 50' DIAM. 80' HIGH FRAMED STEEL BASE 100' HIGH

WITH PIPES OF STAND PIPES AND THEIR DESIGN BY THEODORE C. CURTIS

PLATE No. 7

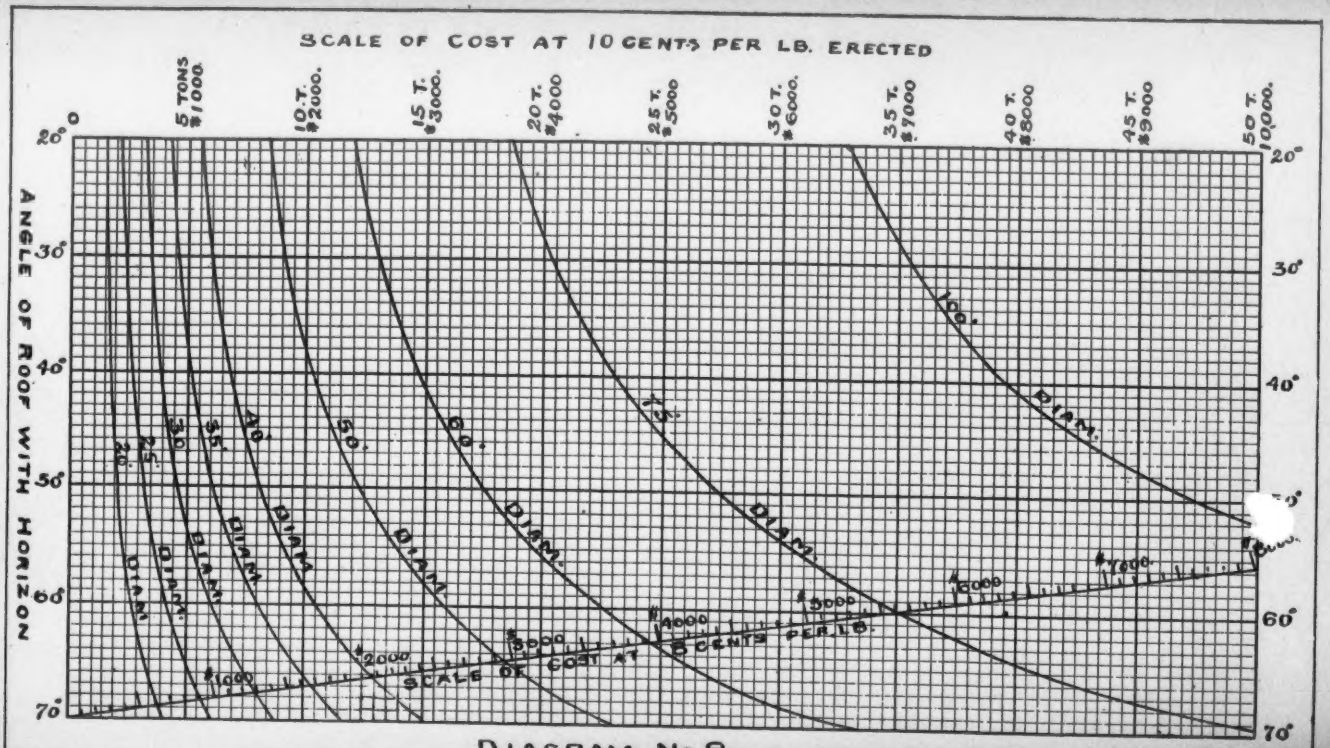
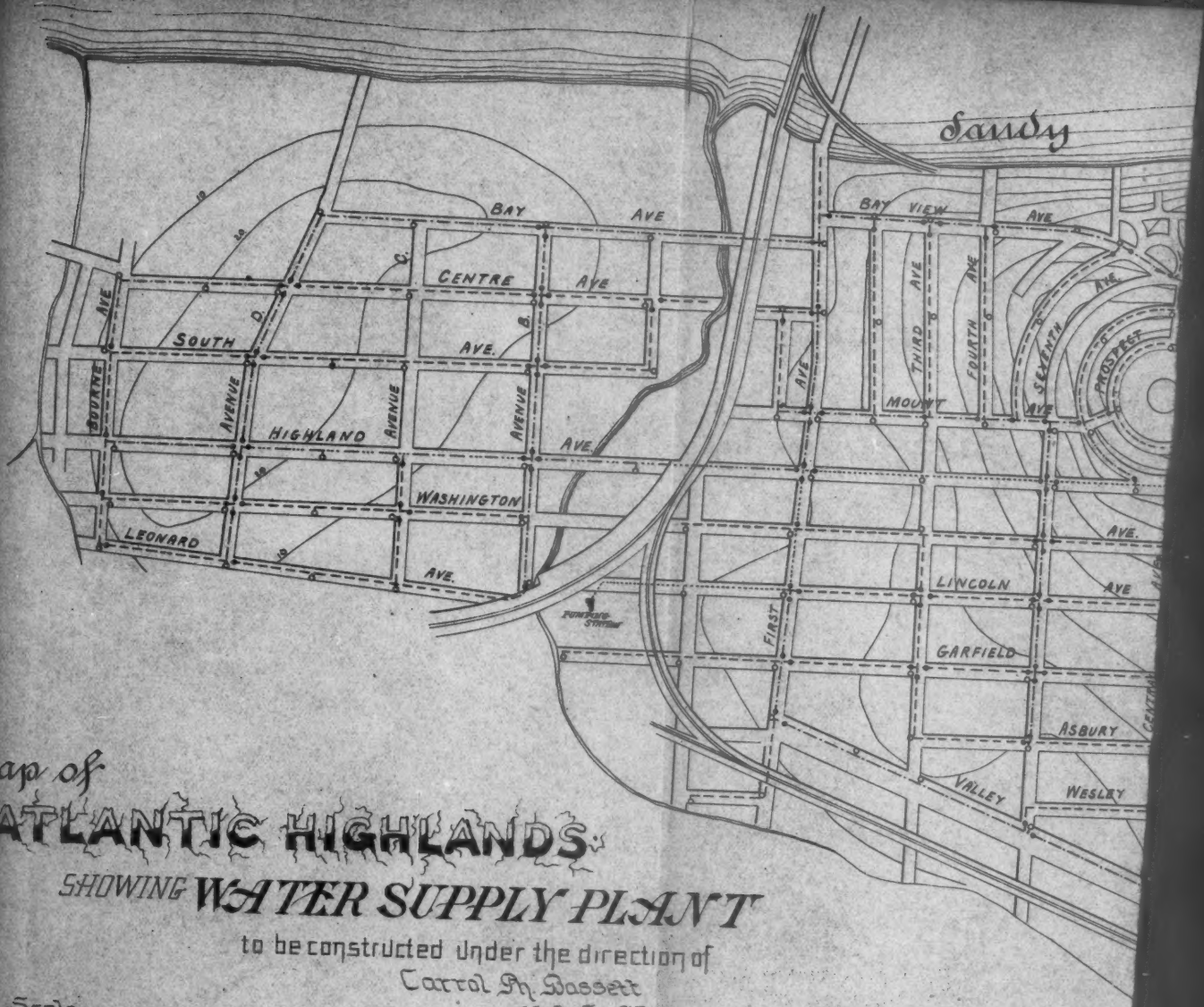


DIAGRAM No 8

COST OF ROOFS FOR STAND PIPES OF DIFFERENT DIAMETERS

WITH PAPER BY FREEMAN C. COFFIN



Map of ATLANTIC HIGHLANDS

SHOWING WATER SUPPLY PLANT

to be constructed under the direction of

Carrol Fr. Bassett

M. Am. Soc C.E.

Scale

10in. pipe .....

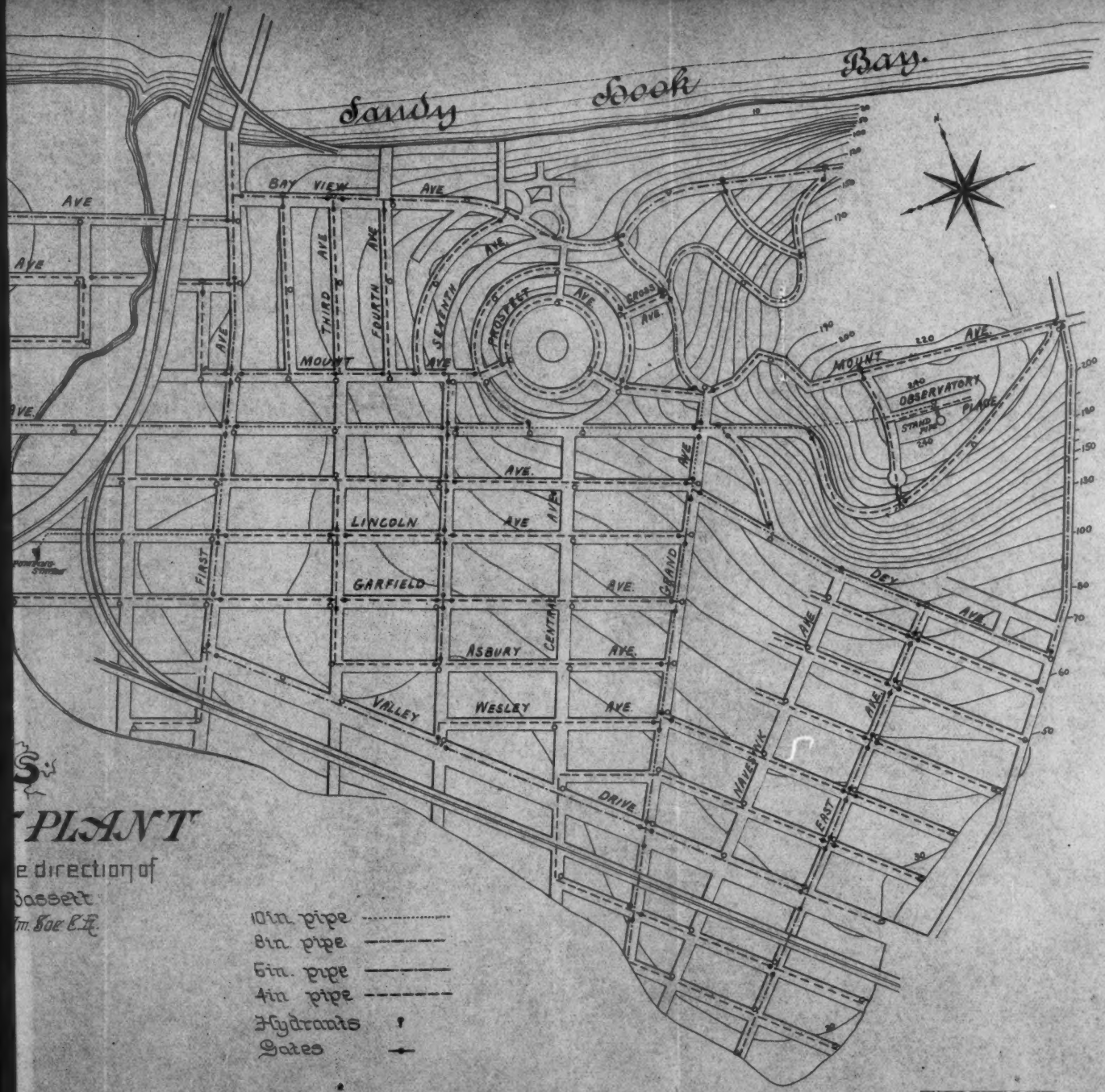
8in. pipe ———

6in. pipe ———

4in pipe - - - -

Hydrants

Gates



PLANT

e direction of

Bassett

Tm. Boe C.E.

CENTRE AVE
SOUTH AVE
HIGHLAND AVENUE
WASHINGTON AVENUE
LEONARD AVENUE
GARFIELD

A HIGH AND LOW SERVICE STAND PIPE AT ATLANTIC HIGHLANDS, NEW JERSEY.

DESIGNED BY C. P. BASSETT, C. E.

With a Short Description of the Water Supply System.

Contributed to the New England Water Works Association by C. P. BASSETT, Chief Engineer. Prepared and presented by A. P. Folwell, Resident Engineer.

The Water Supply system which it is the intention to describe briefly in this paper is situated at Atlantic Highlands, N. J., a borough on the southwest shore of Sandy Hook bay; and for an adequate understanding of the reasons for some of the features of the design a general knowledge of the location and topography is necessary.

From the northern to the southern limits of New England's coast line mountains and hills extend or send their lesser spurs down to the ocean's very edge; Cape Cod being a notable exception—and even this has a considerable hill at its extreme end. But along the Middle and Southern states the flat monotony of the shore is relieved by very few such picturesque features. The traveler by water coming from the south feels a disappointment in the retiring nature of America's grand scenery; until suddenly there rises into view, above the low lying mists which half conceal the "Hook," a beautiful wood-clad hill—the Highlands of Navesink. No less pleasing than this view from the water is the view of the ocean from this high land, a watch hill which is unique on the coast for the attractions it offers as an observatory and for the beautiful and charming picture of water, smooth and rough, of wooded hills, fertile valleys and prosperous villages which meet the sight on all sides. These highlands are extremely abrupt in both their eastern and western slopes, and it is upon and at the foot of the latter that the town of Atlantic Highlands has been built. The map has been provided with contours taken 5 feet apart, from which it may be seen that in a distance of 1500 feet from the water the ground rises to an elevation of 250 feet above low tide (an average slope of over 16 per cent); and that contours within the borough limits vary by as much as 245 feet. A first glance at the town and surrounding country showed the futility of seeking a gravity supply wthin a reasonable distance. There were, moreover, no streams or lakes from which water could be pumped, and the sinking of shallow wells in the neighboring valley was considered; but the water which could be obtained thus was limited in quantity and subject to the objection of possible pollution from population developing upon the contributing water shed. There existed, however, in other New Jersey towns, located over similar geological formation, deep wells furnishing plentiful supplies of water, which suggested that the practicability of an artesian supply be investigated. At Keyport, 8½ miles distant, water

had been found in a sand stratum included between clay marl strata. The state geological survey had found these strata to dip with an incline of from 25 feet to 35 feet per mile. By combining these data, the dip being taken at a mean of 30 feet, it was calculated that water would be reached at a depth of 470 feet in the low-lying western section of the town. Borings were made here and an abundant supply of excellent water found at a depth of 464 feet, in a stratum of sand, and rising 3 feet above the surface. A chemical analysis of this water discovered but one objectionable impurity—iron; which it is proposed to remove by the use of a filter made by the Continental Filter Co.

The highest summit lies at the eastern end of the borough. About 2000 feet west of this is another and lesser one with an elevation of 120 feet above tide water. It would naturally appear, upon a preliminary study of the situation, that these two summits afforded excellent locations for high and low service stand pipes, and this arrangement was considered. There were, however, arguments against this plan and in favor of another which decided the engineer to construct one double stand pipe rather than two separate ones. The lower summit is at the top of an almost circular knoll and is the location of a small private park, all rights in which are held by the owners of the dwellings which entirely surround it. Their consent to the placing of a stand pipe in this park would be given only upon condition that it be an ornamental one provided with an observatory and staircase. And to this the public would be refused access. On the other hand the owners of the land on the eastern and higher summit offered as a gift for the purpose of a stand pipe and reservoir a large lot upon the very summit of the hill, free from all restrictions except that the stucture be ornamental. The high service area to be supplied from this point was small and only a small tank would be required. However the natural and topographical conditions already referred to called for an observatory at this place, and the stipulations of the donors of the land for a more or less ornamental structure. A double expense would thus be called for by two observatories, one of which as such would benefit a very limited number. Should a single stand pipe be built upon the highest land it must of necessity be at least 45 feet high, the water level never falling below 30 feet in order that the high service district be adequately provided for. But, for the low service the head could be reduced several feet—in fact when the increased population warranted it a reservoir would furnish abundant head. It was, moreover, calculated that the cost of the double stand pipe to be described would not greatly exceed that of adding a stairway and observatory to a small high service stand pipe. If it is also kept in mind that but a limited amount of ready money was available, and that anything adding to the attractiveness of Atlantic Highlands to summer visitors would be looked upon in the light of capital, the weight of these arguments is apparent.

For these and other reasons, in place of two separate ones, one double stand pipe upon the higher summit was decided upon and a design made for one to serve both high and low districts, which is believed to be novel and which I will attempt to describe.

Resting upon a suitable concrete foundation, which is brought up to a few inches above the surface of the ground, is the low service stand pipe, directly connected with by far the larger portion of the supply system. This tank is 30 feet in diameter and 35 feet high, having a capacity of 178,000 gallons Resting upon the same foundation and imbedded in the concrete is a hollow cast iron pillar extending vertically through the centre of the low service tank to a height of 30 feet above the foundation. This supports the high service tank, 15 feet in diameter and 15 feet high (having a capacity of 19,800 gallons); the upper 10 feet of it is therefore elevated above the top of the lower tank. Spanning the space between the top of the lower tank and the side of the upper one is a continuous ring of sheet iron, the joints of its plates with each other and with the sides of both tanks being water tight under pressure. In the bottom of this upper tank is a flap valve opening upward. There is an overflow pipe with its funnel mouth a few inches below the top of the upper tank; and a ventilation pipe carried upward from the top plate of the lower tank to a point slightly above the top of the upper one gives free communication with the air and permits the emptying or filling of this tank independently of the other. From the pumps a 10 inch main leads into the lower tank, serving as both inlet and outlet for the low service. Parallel to this for a distance of at least 10 feet from the tank, and connected with it by a 4 inch cross-over is a 4 inch pipe leading up through the hollow cast iron pillar into the upper tank and serving as both inlet and outlet for the high service. Valves are placed in the cross over, and in both the 4 inch and 10 inch mains between it and the stand pipe. After this brief description the working of the tanks can be explained. The details of construction will follow.

Both tanks being empty, the valve in the 10 inch main will be opened and those in the 4 inch main and cross over closed. The pumps being started will then force the water through the 10 inch main into the lower tank. When this has been filled to a depth of slightly more than 30 feet the valve in the bottom of the upper tank will open automatically admitting water into this tank which will in turn be filled to the level of the overflow, when pumping should stop. Meantime the valve in the 4 inch main having been opened, this and the connected service pipes will be filled and both high and low service ready for use.

When the pressure from the pump is lowered the weight of water in the upper tank will close the valve, thus shutting off all connection between the high and low service, as the valve in the cross over has remained closed. It is expected that a short experience in the working of the system will indicate the frequency with which the high service tank must be filled. This will, of course, vary with the increase of population in this district, which from its desirable location, it is expected will soon be completely occupied by the wealthier class of citizens. The area of this high service district is, however, small and it is estimated that even with the fires banked at the pumping station sufficient fire protection will be afforded should the high service tank be kept not less than half full.

When the quantity of water in the upper tank approaches this minimum and it is desired to fill the upper tank before filling the lower, or in case of a fire in the high service district, the valve in the 10 inch main should be closed and that in the cross over opened, when the pump will force directly into the high service main and tank. By closing the valves before referred to in either the 10 inch or 4 inch main either tank can be cut off from the system and cleaned or repaired.

A firm natural foundation of compact sand and gravel is found at the site of the stand pipe. The artificial foundation consists of concrete thoroughly rammed in layers of 6 inches, 36 feet in diameter and 4 feet deep, 3 feet 6 inches being below and 6 inches above the surface. Concrete is thoroughly packed around the inlet, outlet and overflow pipes, and placed to a depth of 5½ feet where the pipes occur and 6 inches wider than the external diameter of the flanges or hubs of the pipes. That part of the foundation under the bed plate of the central column is of portland cement concrete. The concrete is composed of one, two and three parts of cement, sand and broken stone respectively.

The lower stand pipe has six plates in circumference; the upper one, three. The first two plates of the lower stand pipe are 9-16 inches thick, the next two 7-16 inches thick, the next three 5-16 inches thick. The bottom is ⅝ inches thick and the top plate of the bottom stand pipe is ⅝ inches thick supported on angles and braces as shown on the plans.

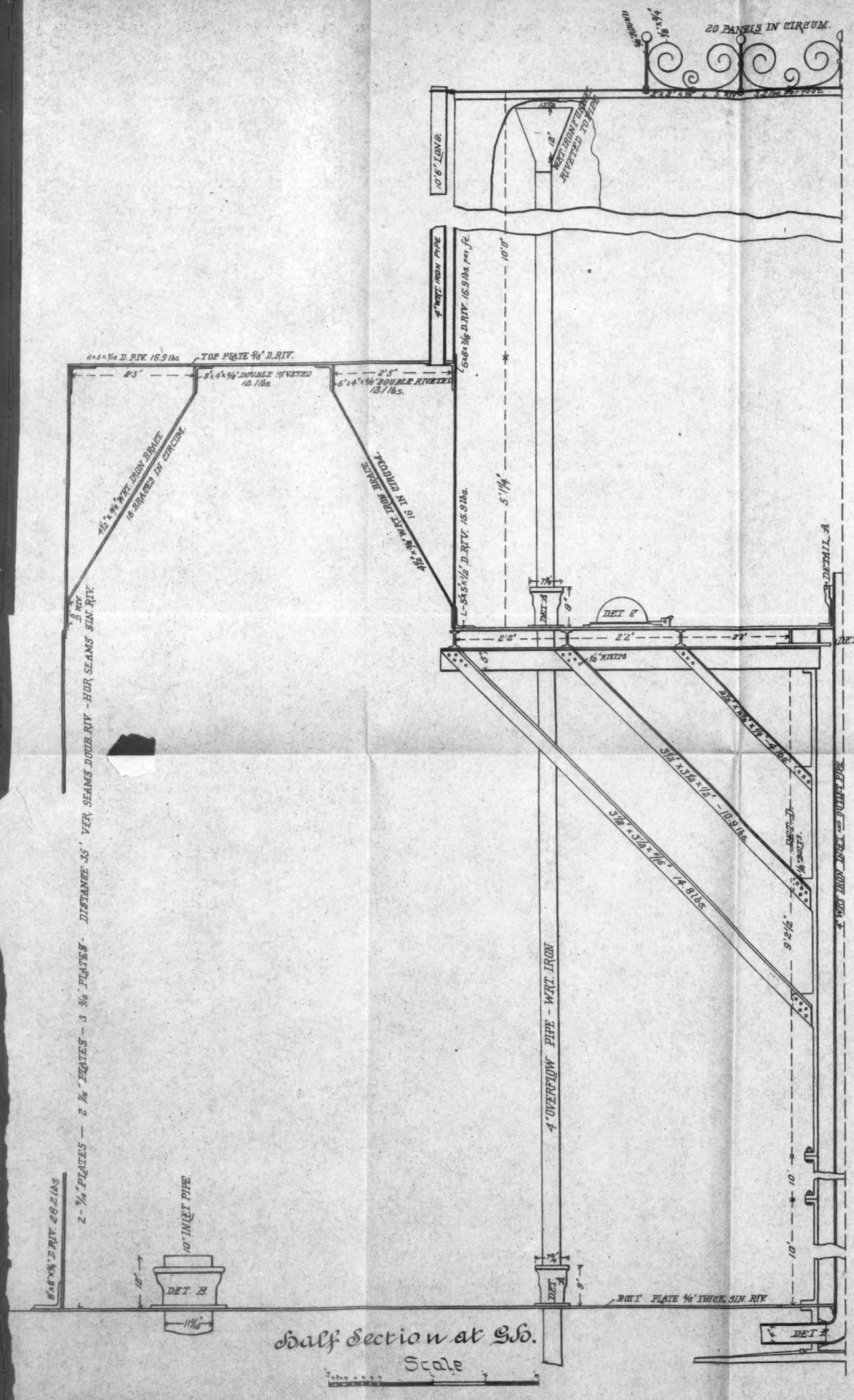
All horizontal seams are single riveted, lap joints; all vertical seams are double riveted, lap joints.

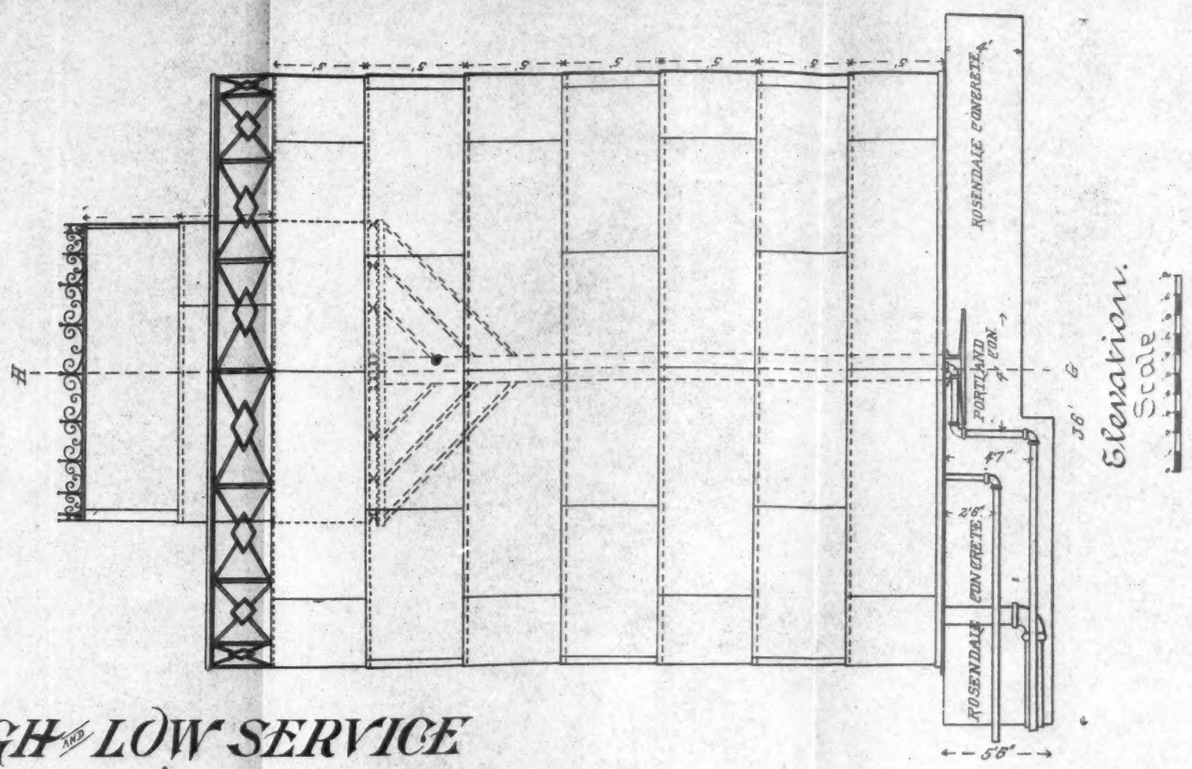
A manhole, 18 inches × 24 inches, is built in a lower sheet of the bottom stand pipe.

All wrought iron is tough, fibrous and of uniform quality; that for plates and angles showing a minimum ultimate tensile strength of 48,000 pounds per square inch.

Supporting the top plate of the lower stand pipe, which acts as the floor of the observatory, are two sets of braces inside the tank, numbering 34 in all. A railing above the sides of this stand pipe surrounds the observatory, standing 3½ feet above the floor.

One foot below the top surface of the foundation, in the centre of the stand pipe, is set a cast iron foot plate 4 feet square, which sustains the weight of the upper tank. The load upon the concrete transmitted by this foot plate when the upper tank is full would therefore be about 80 pounds per square inch. Bolted to and resting upon this foot plate is a hollow cast iron pillar, 1 foot interior diameter and 1 inch thick, rising vertically to a height of about 30 feet above the foundation. Cast on this are three rows of lugs, eight lugs in each row around the pillar, to which are riveted 24 angle iron braces serving to support the beams upon which the top tank rests. These beams are 5 inch steel tees, eight in number, radiating from the cast iron pillar upon which they rest, and connected by a circular wrought iron plate to which they are riveted. Resting upon these beams and forming the im-

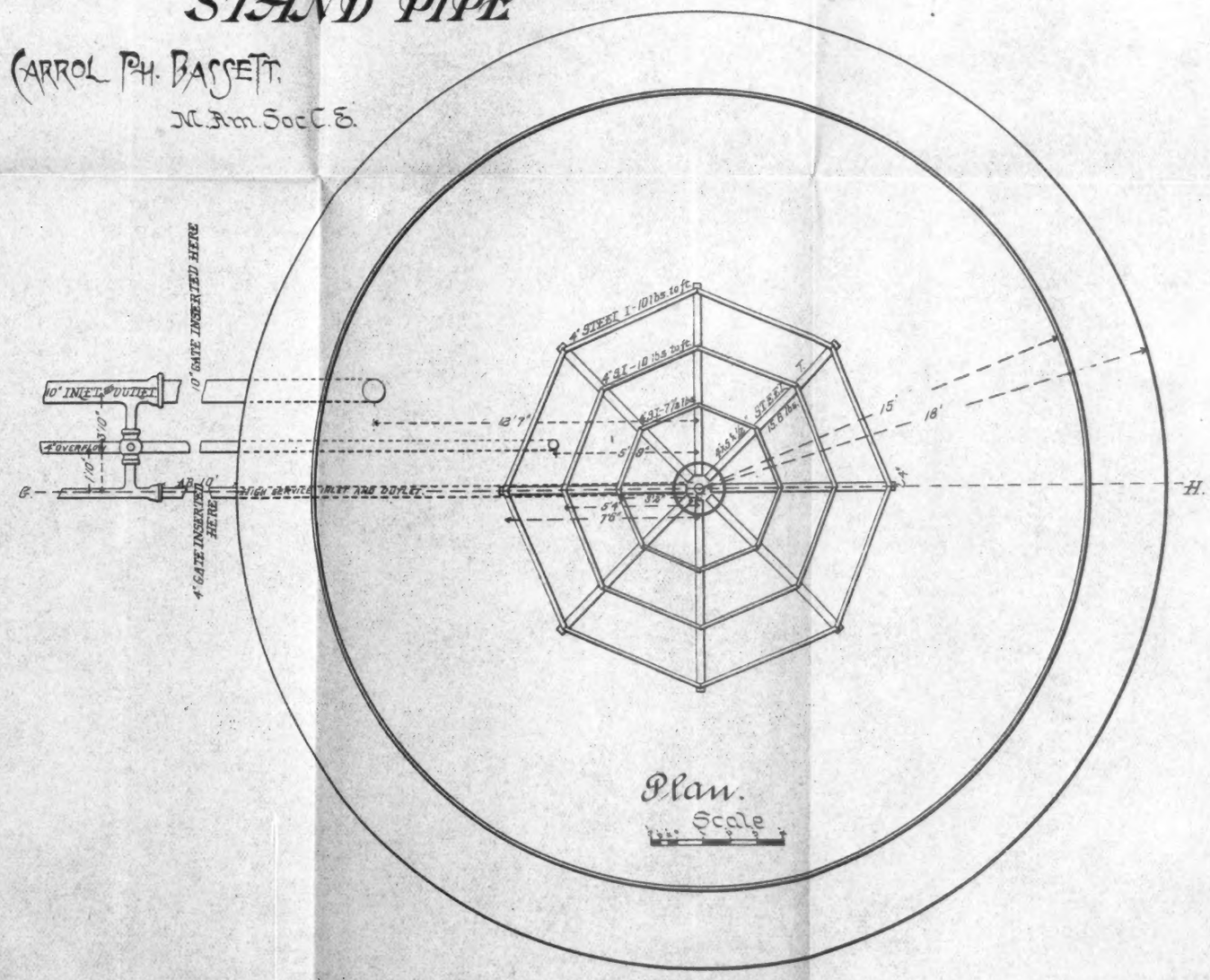

20 PANELS IN CIRCUM.
10'6" LONG.
WRT. IRON TUBES RIVETED TO PLATE
4" WRT. IRON PIPE
TOP PLATE 5/8" D. RIV.
2'5"
2'5"
2'5"
1½"x⅝" WRT. IRON BRACE 18 BRACES IN CIRCUM.
5" WRT. IRON PIPE
16 IN CIRCUIT
DETAIL A
DET. C
DET. E
DET. B
DET. D
DET. F
½" RIVETS
4" OVERFLOW PIPE - WRT. IRON
4" WRT. IRON INLET AND OUTLET PIPE
10" INLET PIPE
BOTT. PLATE ⅝" THICK. SIN. RIV.
Half Section at Gh.
Scale



Elevation.
Scale

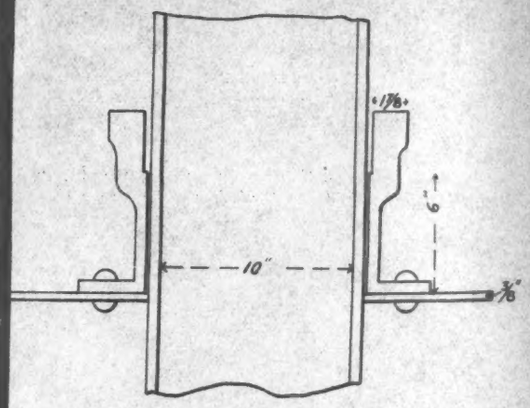
HIGH AND LOW SERVICE STAND PIPE

CARROL PH. BASSETT.
M. Am. Soc. C. E.

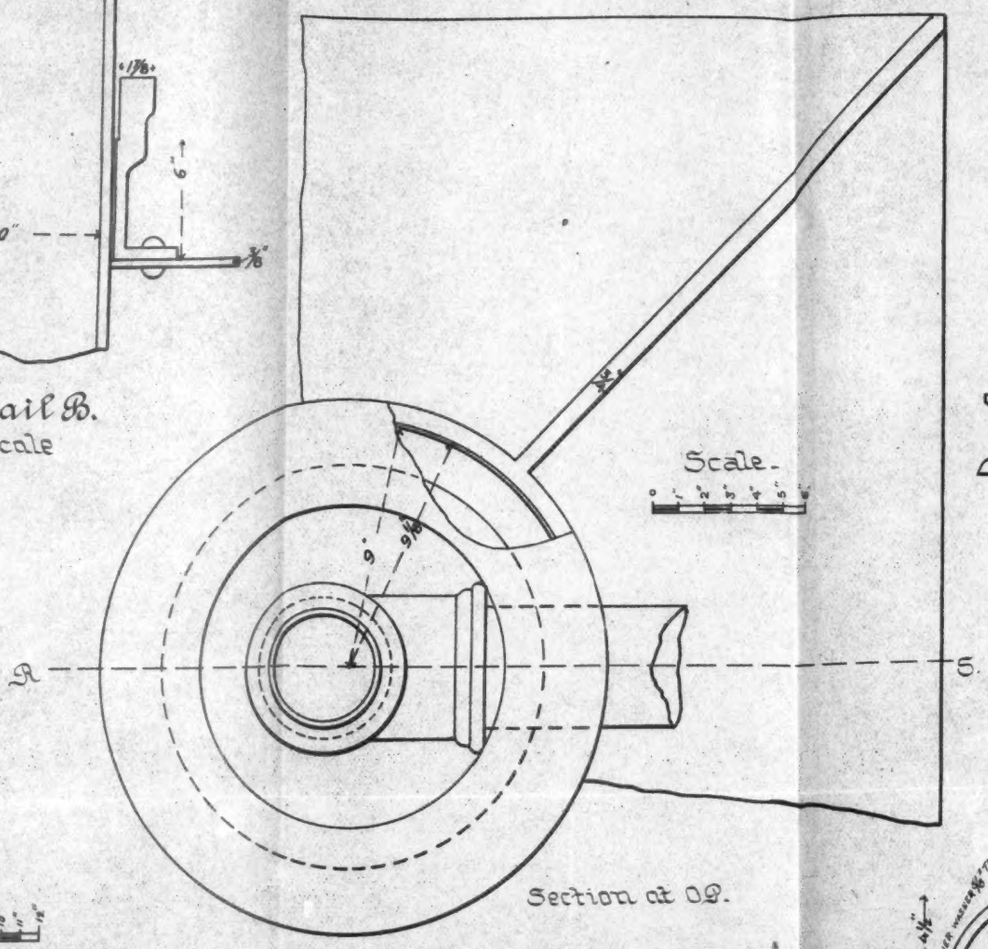


Plan.
Scale

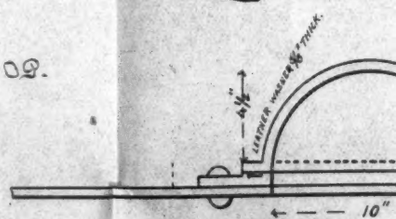
HELIOTYPE PRINTING CO. BOSTON.

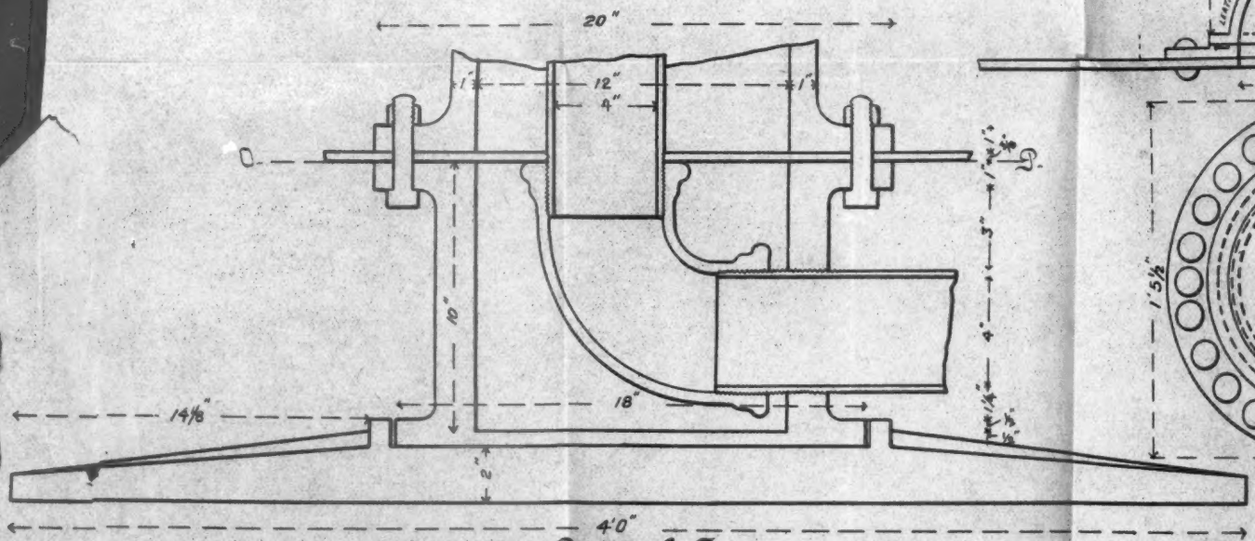


Detail B.
Scale

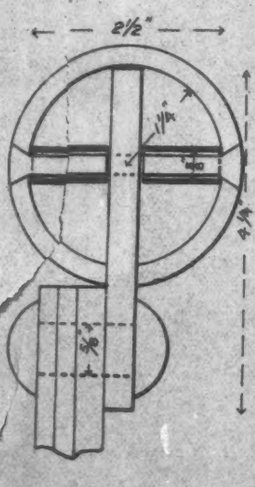


Section at QP.

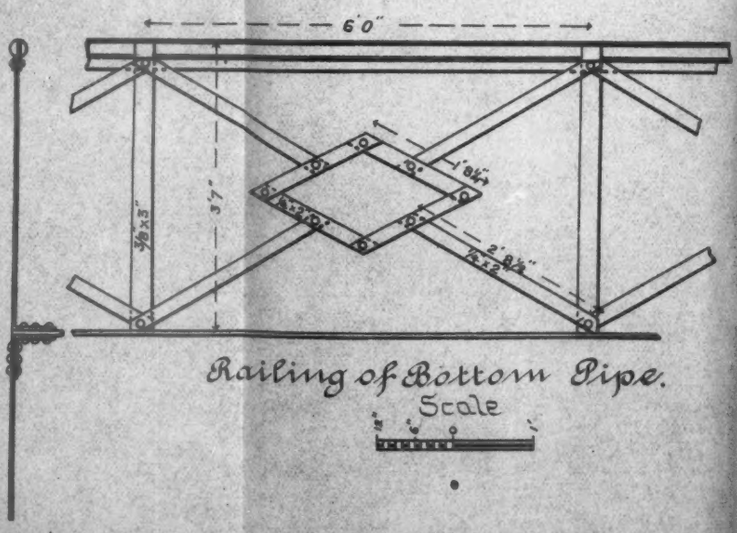




Detail F. Section at RS.

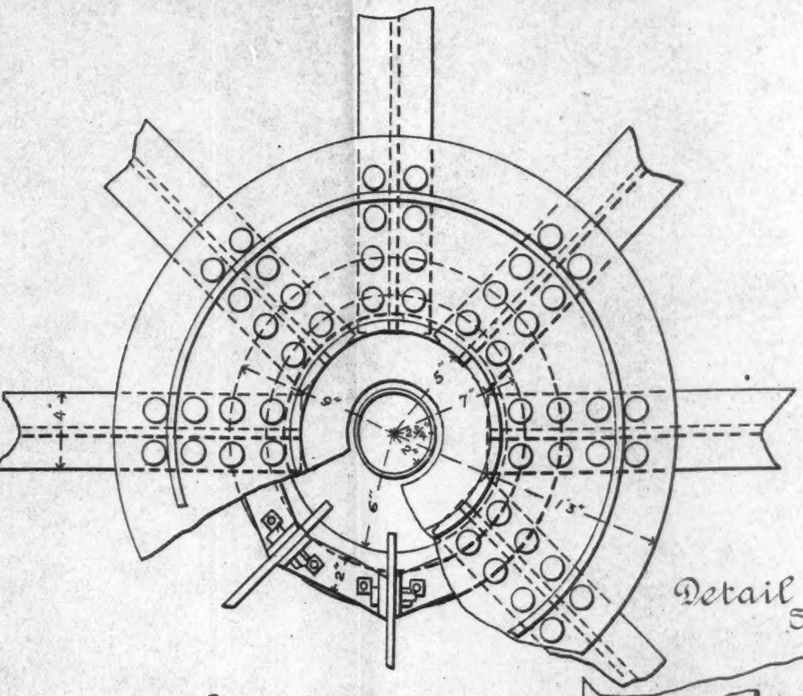


Scale

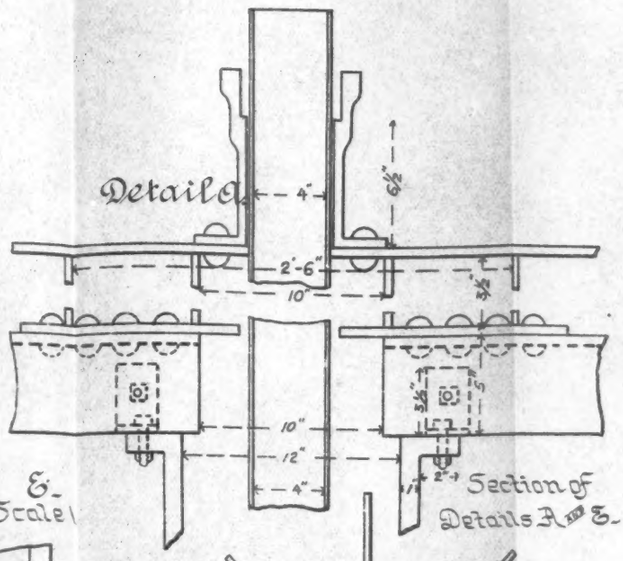


Railing of Bottom Pipe.
Scale

Details of
HIGH AND LOW
ST

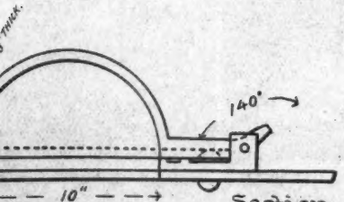


Plan of Detail E.

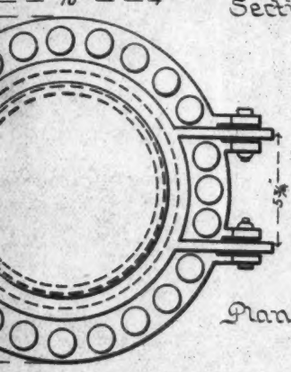


Detail E.
Scale 1

Section of
Details A and E.

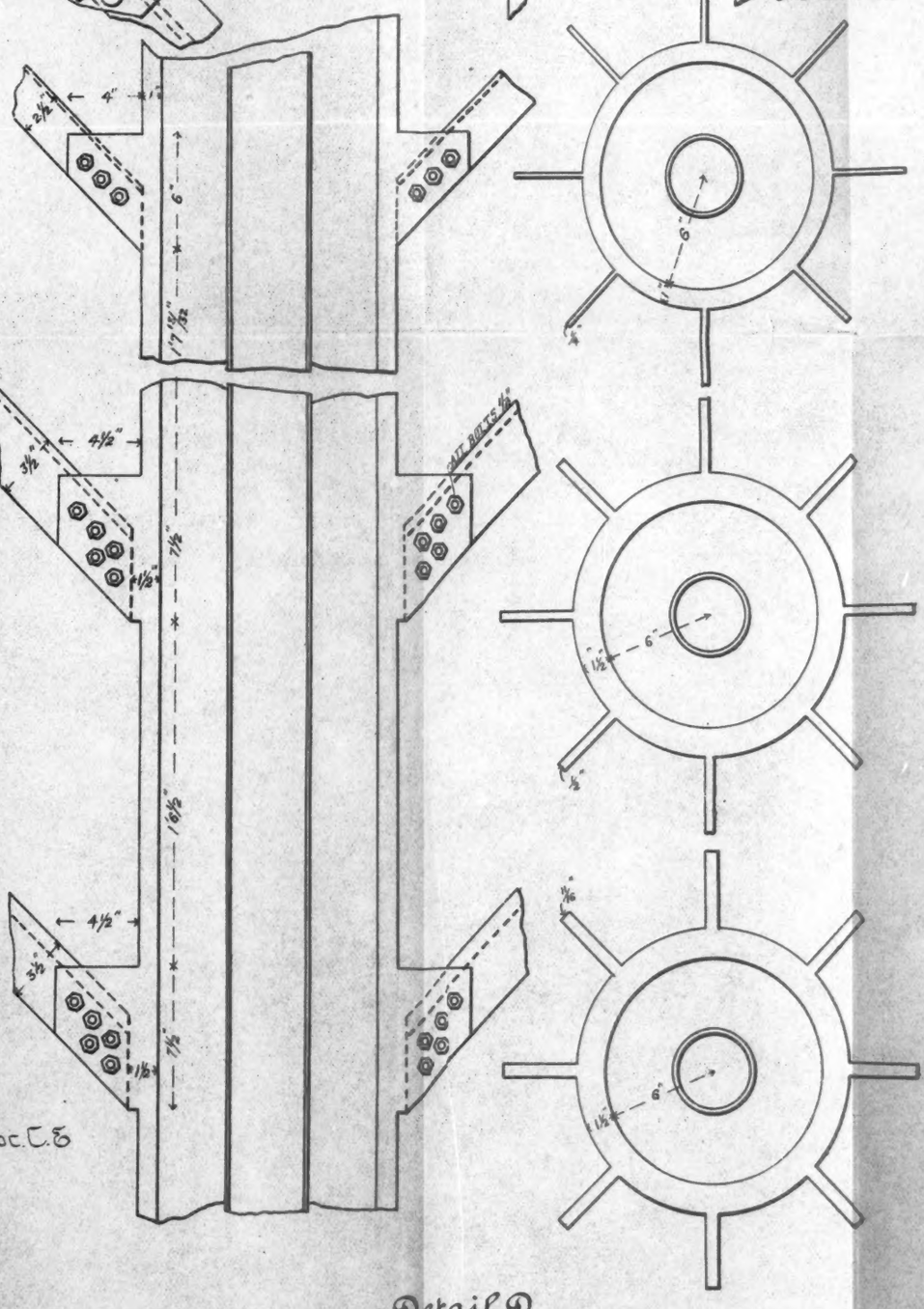


Section



Plan.

Detail C.
Scale 1



Detail D.

LOW SERVICE STAND PIPE

CARROL PH. BASSETT.
M. Am. Soc. C. E.

MELOTYPE PRINTING CO. BOSTON.

mediate support for the upper tank are 4 inch steel I beams spanning the spaces between the T beams and arranged as chords of three concentric circles. The upper tank has a bottom of ¼ inch and sides of ¼ inch iron, is 15 feet high and surrounded by wrought iron fret-work. Riveted to the bottom is a flap valve acting upward, the circular orifice having a diameter of 10 inches. A 4 inch overflow pipe which leads to the sewer extends through the bottoms of both tanks, and is topped by a funnel whose mouth is 13½ inches in diameter and 6 inches below the top of the upper tank. Where this and all other pipes pierce the bottom of either tank a water tight connection is made by means of a cast iron flange surrounding the pipe, riveted to the bottom plate and caulked with lead around the pipe.

The 4 inch inlet pipe for the upper tank is laid through the concrete foundation and rises inside the cast iron pillar to a distance of a foot above the bottom of the tank. This serves also as the outlet pipe. A 4 inch pipe pierces the top plate of the lower tank and rises to a few inches above the top of the upper tank. This allows for escape and admission of air as the water rises or falls in the lower tank.

The 10 inch pumping main, rising a foot above the bottom of the lower tank also serves as both inlet and outlet pipe.

As the stand pipe is placed at one side of the borough, while the pump is near the centre; and as the pumping main also serves as a distribution main, this is practically a direct pumping system; the stand pipe, in its character as a reservoir, acting mainly as a reserve for fire and other emergencies, and for supplying the slight night consumption, when the pumps will be idle. With this condition in view the pumps are made with long stroke and consequent low speed, the latter being at a rate of 60 feet per minute. The entire lift of the pumps from the wells to the top of the stand pipe is 300 feet. Calculation is made on a basis of a present daily consumption of 750,000 gallons; but provision is made for duplicating the pumping plant in the future.

An area of ground adjacent to the stand pipe is held sufficient for a reservoir should this at some time be needed; in which case the pumping main would lead directly into the reservoir, and a branch from this main connect with the stand pipe. The necessity for this additional storage may be created by future extension of population into districts south and east of the stand pipe which are outside the present borough limits.

LAYING A MAIN ON A RIVER BOTTOM. THAWING FROZEN SERVICE PIPES. PLUGS.

An Experience Talk.

BY

CHAS. K. WALKER, Superintendent, Manchester, N. H.

In laying an 8 inch main across the Merrimac river I adopted, by the advice of a contractor and contrary to better judgment, a pipe ¾ inch thick with a bell 3 inches deep, but it proved an utter failure, breaking in two the first winter and pulling apart the next year. As a substitute an 8 inch main was laid across the bridge and left exposed, and it stands today all right. My experience is this: If I was to lay a pipe across the river in the same place today, I would get one an inch thick and have the joints similar to those used in Boston. I would have a 5 inch bell, and three set screws to each joint. I wouldn't connect it on either side until it had settled a week or two, and I would drive those joints and run the lead beyond the set screws, and I wouldn't be afraid to drop that pipe anywhere in any river or on any kind of bottom.

We have had a little experience in thawing out service pipes. We have tried various ways, and now we dig a hole two feet deep and about three feet across, we put in some lime and pour on some water, go home and go to bed, and the next morning everything is all thawed out.

Now, plugs. Did you ever have any trouble with plugs? They are about ten times as heavy as they ought to be, and when you come to break them out you can't do it without much trouble. I have had lighter ones made, about half or three-quarters of an inch thick that worked all right. But the best plug is the old plug they take out of cement pipe, and that is all there is good about a cement pipe, the plug.

THE LIFE OF SERVICE PIPES.

An Experience Talk.

BY

Mr. F. L. Fuller, Civil Engineer, Boston.

The Wellesley Water Works were built in 1884, service pipes were put in during the fall, the next spring and summer, and the kind of pipe adopted was plain tarred pipe. It was necessary to hurry to get them in, especially in the fall, and that seemed to be the most available pipe, so the services were put in of tarred iron pipe without any lining. During the past year we have added a good many meters to the system, and some of those pipes have been cut. In 1886 or 1887 we changed from the iron pipe to the cement lined pipe, and what I wish to present to you today is the condition of some of those first service pipes which were put in. I have a few samples here, and perhaps you may be interested to see them. Most of them have been in from six to seven years, and by looking at them you can see what sort of a condition they are in. (The speaker exhibited samples of several kinds of pipe.)

I had some curiosity in my own house to know about what the flow would be through an ordinary service pipe, and so I put 50 feet of rubber hose on to a faucet at my sink, and carefully measured the flow of that water. This service pipe was put in in the spring of 1886, and in November of that year I measured the flow through that pipe. I put on this piece of hose and carefully measured a wash tub and saw how long it took to fill this wash tub under those conditions, and the time, as I recall it was one minute and five seconds. I thought this last winter I would repeat the same operation, and with exactly the same piece of hose and the same wash tub. I found the time required was seven minutes and fifty seconds; so you can judge what the condition of the service must have been, the time varying from one minute and five seconds in 1886 to seven minutes and fifty seconds in 1893. The flow in the first instance was about 15 gallons a minute, or a little over; this year it was two gallons a minute.

Mr. FitzGerald. The same pressure on the main?

Mr. Fuller. Just the same conditions exactly. The pressure at that point is somewhere in the neighborhood of 70 pounds. It seems to me any town is very foolish to put in either plain pipe or tarred pipe. It is much better to put in cement lined pipe, or possibly the lead lined pipe, although we haven't had any experience with that. But this experiment of measuring the flow of that pipe after such an interval of time convinces me that plain iron pipe is not a proper pipe to put in for services.

LOCATION OF BROKEN MAINS AND EXAMINATION OF HYDRANTS IN COLD WEATHER.

An Experience Talk.

BY

GEORGE A. STACY, Superintendent, Marlboro, Mass.

Mr. President, what little I have to say today will give you my experience in the discovery of a break in a 16 inch main when there was about three feet of frost in the ground. We had just laid this main, and it had been under pressure about three weeks when word came that there was something the matter with the pumps and I was wanted up there immediately. I drove up expecting to find that something had broken. About 500 feet from the station, in a slight depression in the street, I saw quite a lake of water. We shut off the small district which was supplied by this force main, and then waited until the water subsided in the street, when we expected to find an upheaval in the ground large enough to indicate where the break had occurred; but to our surprise we couldn't find any place where the ground wasn't as hard as before. After a little investigation we found that the water had come up from a culvert, which crosses the road at the lowest point in the depression. It was evident that the water had forced its way from the break through the ground into this culvert in such a volume that it could not discharge into the lake which was immediately on the other side, and so had come up out of the culvert, which was about eight inches deep, and had filled the road. Now, the question was, where was the break? We could not tell which side of the culvert it was, and as there was three feet of solid frost in the ground, how were we going to find the break without digging the whole length? We couldn't get into the culvert to see which way the water was running, and we didn't want to go to work blindly, for the break might be 100 feet either side of the culvert, or it might be close to it.

The method I adopted to locate the break was as follows: With a common stone drill, we drilled through the frost at one point and found practically dry earth under the frost there; then we went up beyond the culvert about 100 feet and found practically dry earth under the frost there; so I was satisfied the break was somewhere between those two points. At the fourth drilling, as soon as the drill went through the frost it dropped, and we found plenty of water, and when we dug down I found we were within two feet of the fracture. This is perhaps a small matter to speak about, but I thought I could mention it here, for possibly my experience might save somebody some trouble in the future if he should happen to find himself in a like situation. It saved me a great deal of labor, and I found it very easy to locate the leak

in this way. I might add that the break was from 25 to 30 feet to the west of the culvert. I found by drilling these holes that the water had washed out the dirt as it flowed towards the culvert so it told you very readily whether you were nearing the break, and by dividing up the distance it didn't take a great while to come close to the point, and we struck within two feet of it when we drilled the last hole. We then went immediately to work and made the repairs.

During the severe weather this winter we have had good fortune in Marlboro in regard to our hydrants. I know some people advocate the inspection of hydrants in the fall and not touching them again during the winter except in case of fire. But I don't see how it is possible to follow that plan with any certainty of safety. I know the theory is that if a hydrant is all right in the fall and then is not disturbed, it will remain all right, but I do not find it so in my experience. During the past winter we have found four hydrants out of 281 frozen, and one case occurred about a month ago, a hydrant that has been set ever since 1883 and had naver before given us trouble. We knew it was properly drained, the drip was clear, and it always had worked free. It had been examined every 48 hours in cold weather, and every 24 hours in extreme cold weather, as we examine every hydrant in the city, but yet one morning about 8 o'clock one of my men found it stuck. We went up there with the pump and thawed it out, finding the ice was within a foot or six inches of the surface of the street. It was quite a mystery for a while how the water had got in there. The hydrant was set near an embankment where the natural slope of the ground water was away from it, it is set extra deep, and we never had had any trouble whatever with it. The mystery was explained two days afterwards by finding a leak in a service pipe which tapped the main within two feet of the hydrant branch. The water had forced its way along the pipe, the frost being so deep it could not come up through the ground, and consequently it put pressure enough in the ground to fill the hydrant post up within a foot or six inches of the surface. Now, no man could tell whether that hydrant was frozen or not unless he examined it. It was all right in the fall, it is all right today, and it was all right the minute that leak was repaired. This is why I say no man can feel safe about his hydrants if he does not examine them frequently in cold weather; and this experience shows the danger even under a system of inspection, for this hydrant had been examined certainly within 48 hours, and was found to be all right. I had another experience where the water followed a service pipe down through a ledge which was blasted out to set the hydrant and where we had run the pipe up through the hydrant trench to avoid extra blasting, it followed down through there and got into the hydrant post and froze. These little things are occurring all the time, and so "eternal vigilance" is necessary during cold weather, and even then, when you are trying to do your very best, you are liable to get caught and get blamed for it. It may do in some places, but certainly it won't do for me to sit down and say the hydrants are all right in the fall, and therefore they are going to remain so without further attention until spring, unless there is a fire.

THE PRESIDENT. I suppose Mr. Stacy doesn't make such an examination as some of the firemen would like to make, by putting on a hose and opening the valve.

MR. STACY. If the hydrant is all right I don't believe, of course, in disturbing it. I believe hydrants should be made so they can be tested without disturbing them. I can test my hydrants as fast as I can walk along the street and be absolutely sure they are all right, for the reason that if the supply pipe is not frozen, the water in the bottom of the hydrant will not be frozen, and the spindle will be free. If the water in the supply pipe is frozen it will grip the spindle and hold it as rigidly as in a vise. The back lash in the nut of the valve permits the movement of the spindle to a slight extent, and by that method we can determine whether the hydrant is free or not.

STATISTICS

FOR

THE YEARS 1888—1892.

IN FORM ADOPTED BY THE

New England Water Works ASSOCIATION.

TABLE I—GENERAL AND PUMPING.

TABLE II—FINANCIAL.

TABLE III—CONSUMPTION.

TABLE IV—DISTRIBUTION—MAIN PIPE.

TABLE V—DISTRIBUTION—SERVICE PIPE.

Compiled by
SENIOR EDITOR,
Boston, Mass.

TABLE I.—GENERAL

Statistics for the Years 1888–1892. In Form Adopted

Compiled by the Seni

Name of Place.	Year.	Date of Construction.	By whom owned.	Source of Supply.	Mode of Supply.	1. Builders of Pumping Machinery.	2. Coal Used.	Per cent ashes.	Price
Boston	1888	1848	City	Lake Cochituate and Sudbury River.	Gravity and pumping to Reservoir	Holly Man. Co.	Bituminous	7.3	$
"	1889	"	"				"	7.9	
"	1890	"	"				"	8.2	
"	1891	"	"				"	8.5	
"	1892	"	"				"	8.5	
Boston	1888	1864	"	Mystic Lake	Pumping to Reservoir	H. R. Worthington	Bituminous	8 4	4.
"	1889	"	"	"			"	8.0	
"	1890	"	"	"			"	9.8	
"	1891	"	"	"			"	10.2	
"	1892	"	"	"			"	10.2	
Burlington	1888	1867–68	City	Lake Champlain	Pumping to Reservoir	H. R. Worthington	Anthracite and Mill Shavings		
"	1889	"	"	"			"		
"	1890	"	"	"			"		
"	1891	"	"	"			"		
"	1892	"	"	"			"		
Fall River	1888	1874	"	Watuppa Lake	Pumping to Stand Pipe and Tank	Botson Machine Co., H. R. Worthington, Davidson Steam Pump Co.	Bituminous	9 6	
"	1889	"	"	"			"	8.5	
"	1890	"	"	"			"	8.4	
"	1891	"	"	"			"	7 6	
"	1892	"	"	"			"	7.3	
Fitchburg		1871–72	City	Storage Reservoirs.	Gravitation				
Holyoke		1873	"	Two lakes and two streams	Gravitation				
Lynn	1888	1870–72	"	Four artificial storage basins.	Pumping to Reservoir	Engine designed by E. D. Leavitt, Jr., built by I. P. Morris, Deane Steam Pump Co., and Loretz Engine Co.	Bituminous		
"	1889	"	"				"		
"	1890	"	"				"		
"	1891	"	"				"		
"	1892	"	"				"	5.4	
New Bedford	1888	1866 69	City	Acushnet River	Pumping to Reservoir	Quintard Iron Works		11.9	
						H. R. Worthington.	Anthracite	10.2	
						H. R. Worthington.			
"	1889	"	"	"		Quintard Iron Works.			
						H R. Worthington.	Anthracite	10.2	
						H. R. Worthington.		9.6	
"	1890	"	"	"		Quintard Iron Works.			
						H. R. Worthington.	Anthracite	9.6	
						H. R. Worthington.		13.2	
"	1891	"	"	"		Quintard Iron Works.			
						H. R. Worthington.	Anthracite	13.2	
						H. R. Worthington.		13.0	
"	1892	"	"	"		Quintard Iron Works	Anthracite	13.0	
						H. R. Worthington.		13.2	
						H. R. Worthington.	Bituminous		
New London		1872	City	Lake Konomoc	Gravity				
Newton	1888	1876	"	Filter Basin near Charles River.	Pumping to Reservoir	H R. Worthington	Bituminous		
"	1889	"	"				"		
"	1890	"	"				"	6 0	
"	1891	"	"				"	6.0	
"	1892	"	"				"	6.0	
Plymouth	1888	1855	Town	Great and Little South Ponds and Lout Pond.	Low service Gravity. High service pump to Reservoir.	H. R. Worthington	Bituminous		
"	1889	"	"				"		
"	1890	"	"				"		
"	1891	"	"				"		
"	1892	"	"				"		
Springfield		1864–90	City	Storage Reservoir.	Gravity				
Taunton	1888	1876	"	Ground water and Taunton River.	Direct Pumping	Holly Man. Co.	Anthracite	13.0	
"	1889	"	"				"	13.0	
"	1890	"	"				"		
"	1891	"	"				Ant. & Bit'ms		5.1
"	1892	"	"				Bituminous	13.0	
Troy	1888	1833	City	Five Storage Reservoirs in valley of the Piscawankill River, also the Hudson River.	Gravity and Pumping	Holly Man. Co.	Anthracite		2.57, 2.4
							Bituminous	18.5	3.6, 4.2
"	1889	"	"				Anthracite		2.37
							Bituminous	17.9	3.42
"	1890	"	"				Anthracite		2.21
							Bituminous	19 7	3.31
"	1891	"	"				Anthracite		2 11
							Bituminous	18.3	3.25
"	1892	"	"				Anthracite		2 01
							Bituminous	18.0	3.20, 3.8
Waltham	1888	1872–73	City	Filter Basin near Charles River.	Pumping to Reservoir.	H. R. Worthington	Anthracite	12.7	
"	1889	"	"				"	11 9	
"	1890	"	"				"	11.9	
"	1891	"	"				Bituminous	14 0	
"	1892	"	"				"	14 0	
Ware	1888	1886	Town	Collecting Well	Pumping to Reservoir	Deane Steam Pump Co.	Bituminous		
"	1889	"	"	"			"		
"	1890	"	"	"			"		
"	1891	"	"	"			"		
"	1892	"	"	"			"		
Wellesley	1888	1884	Town	Two Wells	Pumping to Reservoir	H. R. Worthington.	Bituminous	9.0	
"	1889	"	"	"			Bit'ms&Sc'n'gs		5.25, 2.6
"	1890	"	"	"			"	9.0	5 25, 2.5
"	1891	"	"	"			"	9.0	5.25, 2.5
"	1892	"	"	"				9.5	
Woonsocket	1888	1884	Town	Crooks Fall Brook	Pumping to Tank	G. F. Blake Man. Co.	Bituminous	7.8	4.70, 5.1
"	1889	"	"	"			"	7.6	4 90, 5.1
"	1890	"	"	"			Bit'ms & Ant.	10.1	4.50, 4.9
"	1891	"	"	"			Bituminous	7.3	4 50
"	1892	"	"	"			"	7.0	4 53

ERAL AND PUMPING.

Adopted by the New England Water Works Association.

by the Senior Editor.

2.		3.	4.	5.	6.	7.	8.	9.	10.	11.	12.	13.	14.
Per cent ashes.	Price per ton of 2,240 lbs.	Coal consumed for year, lbs.	Lbs. of Wood. 3.	Total fuel consumed for year, lbs.	Total pumpage for the year in gallons.	Average static head against pumps.	Average dynamic head against pumps.	No. of gallons pumped per lb. of coal.	Duty in ft., lbs. per 100 lbs. coal, no deductions.	Cost per million gallons pumped into reservoir. Fig'd on pumping station expenses.	Cost per million gallons raised 1 ft. high. Figured on pumping station expenses.	Cost per million gallons pumped into reservoir. Figured on total maintenance.	Cost per million gallons raised 1 ft. high. Figured on total maintenance.
7.3	$ 4.98	2,626,558		2,626,558	1,805,374,800		126.4	687.	72,459,200	$ 8.69	$ 0.069	$	
7.9	4 78	2,462 750		2.462,750	2,143,925,800		123.2	870.5	95,445,400	7.86	0.064		
8.2	4 70	2,677,281		2,677,281	2,369,631,700		123.2	885.	98,069,200	7.61	0.062		
8.5	4.90	2,910,751		2.910,751	2,651,164,400		124 6	911.	101,380,800	6.65	0.053		
8.5	4.66	3,548,105		3,548,105	3,050,018 300		126.3	944.	99,195,300	6.15	0.049		
8 4	4.23, 4 29	6.924,000		6,924,000	3,022,322.400		147.62	436.	53,750,600	8.07	0.055		
8.0	4 88	6 286,000		6,286,000	2.857,969.400		146.5	455	55,554,200	8.89	0.061		
9.8	4.20	6,506,000		6,506.000	3,030,116,500		147.1	465.	57.141,800	7.76	0.053		
10.2	4.34	6,988,500		6,988.500	3,304,951.000		148.	473	58.380,500	7.85	0 053		
10.2	4.13	7,873.000		7,873,000	3,596,716 400		148.8	457.	56,709,000	7.09	0.048		
	4.65				254,360,275	289.	316.			21.58	0.068	98.11	0.31
	4.75				257,558,200	289.	316.			24·57	0.078	103·49	0.33
	4.85				279,371,350	289.	316.			21.52	0.068	98.83	0.312
	4.85				298,500,575	289.	316			23.13	0·073	104.92	0.332
					288,090,575								
9 6		2,022,935		2,022,935	647,279,612		185.5	320.		10.70		185.74	
8.5		2,051,920		2,051,920	685,447,036		184.9	334.		12.47		178.50	
8.4		2,339,435		2,339,435	779.706,398		184.9	333.		12.07		157.75	
7 6		2,637,333		2,637,333	859,830,568		185.9	326.		12.19			
7.3		2,588,420		2.588.420	836,656,878		187.0	323.		11.77		153.18	
	5.49	1,398,105		1,398,105	906,397,617		168.6	641.1	101,831,525	9.12	0.054		
	5.54	1,295,522		1,295,522	895.966,403		170.2	691.6	106,628,900	8.65	0.05		
	5.32	1,236,165		1,236,165	967,336,364		172.4	782.	116,930,878	7.65	0.044		
5.4	4.98	1,739,483			1,299,041,098		158.8	760.4	100,789,536	7.49	0.047	75.45	0.47
11.9	5 48	812,135	634	812,769	471,008,048	125.7	136.1	580.	65,764,214				
10.2	5.89	494,912	267	495,179	227,035,848	125.	128.5	459.	49,123,434	8.83	0.067	62.94	0.48
		820,193	266	820,459	533.954,736	125.3	127.4	651.	70,340,235				
		736,914	367	737.281	446,583,568	125.7	135.0	606.	68,202,718				
10.2	5.89	154,700		154,700	74,535,760	124.9	126.2	482.	50,709,050	8.04	0.062	56.59	0.435
9.6	5.44	1,138.515	300	1,138,815	777,506,410	125.3	127.6	683.	72,689,510				
		1,242,730	585	1,243,315	755,911,579	125.2	134.5	608	68,199,024				
9.6	5.44	220,935	405	221,340	109,852,408	124.8	126.0	497.	52,166,255	7.12	0.054	50.31	0.384
13.2	4.50	792,350	270	792.620	597,191,364	125.	127.4	754.	80,022,754				
		1,271.900	650	1,272,550	774,010,775	125.6	134.9	609.	68,410,278				
13.2	4.50	191,000	67	191.067	91,483,092	124.7	126.0	479.	50,298,363	6.99	0.053	62.19	0.474
13.0	5.15	918,679	333	918,697	659,290,958	125.3	127.6	718.	76,375,766				
13.0	5.15	934,150	200	934,350	584,860,523	125.8	135.1	626.	70,523,414				
13.2	5.00	428,100	267	428,367	214,567,440	125.1	126.3	501.	52,757,332	6.90	0.053	48.23	0.37
	4.00	1,095,680	267	1,095,947	792,588,940	125.6	127.9	723.	77,160,824				
	5.10	1,024,600	21,952	1,046,552	256,031,474	176.1	230.1	274.	52,560,574	23 20	0.10	287.23	1.25
	5.25	1,289,600	19,200	1,308,800	315,370,007	176.8	227.7	241.	45,758,964	16.85	0.07	243.40	1.07
6 0	4.87	1,340,900	12,800	1,353,700	359.487,217	176.6		266.		18.85		227.10	
6.0	4.90	1,542,700	13,000	1,555,700	388,753,278	199.6		252.		13.83		226.71	
6.0	4.63	1,410,200	13,000	1,423,200	463,686,000	235.	235.	325.	63,854,000	13.39	0.057	203.56	0.866
	4.90	196,275		196,275	94,992,542	64.	66.	485.	26,675,000	12.59	0.189	110.07	1.67
	4.89	199,055		199,055	94,721,616	65.	66.	476.	26,695,000	17.13	0.233	112.44	1.73
	4.95	209,460		209,460	100.124,640	65.	66.	478.	26,311,582	17.88	0.27	104.70	1.58
	4.80	219,250		219,250	99,863 280	65	66.	455.	25,070,890	19.83	0.30	113 47	1.72
13.0	5.58	812,000		812,000	275,188,656		153.	339.	42,244,609	19.45	0.126	146.41	0.96
13.0	5.27	817,882		817,882	280,885,963		152.2	343.	43,381,403	12.09	0.079	138.99	0.91
	5 15	819,400		819,400	290,801,368		152.4	355.	45,107,785	18.68	0.122	138.13	0.90
	5.15, 4.60	929,100		929,100	374,107,200		152.5	403.	50,972,800	15.03	0.098	112.58	0.74
13.0	4.35	910,600		910,600	370,453,491		152.8	407.	51,843,636	14.77	0.097	119.39	0.78
	2.57, 2.43, 2.37	4,578,604											
18.5	3.6, 4.2, 3.42	2,428,227	250	7,006,833	1,741,866,964	231.	242.	252.	51,082,700	13.82	0.06		
	2.37 2.21	4,609,808											
17.9	3.42 3.31	2,117,928	193	6,727,736	1,736,863,103	231.	242.	254.	51,471,826	12.07	0.052		
	2.21 2.11	3,560,517											
19 7	3.31 3.25	1,617,571	2,033	5,180,121	1,306,773,232	231.	242.	252.	50,934,520	14.37	0.062		
	2 11 2 01	5,236,387											
18.3	3.25 3.20	2,657,550	2,033	7,893,937	2,006.391,944	231.	242.	254.	51,298,366	11.60	0.050		
	2 01	8,069,652											
18.0	3.20, 3.80	1,018,571		9.088,223	2,210,887,684	231.	242.	245.	49,098.601	11.75	0.051		
12.7	6.68	713,900		713 900	194,933,158	164.	192.	273.	43,720,512	23.49	0.122	191.32	0.996
11 9	6.72	759.200		759.200	207,809,842	164.	192.	275.	43,830,526	19.49	0.101	150.87	0.784
11.9	6 72	836,000		836,000	228,409,473	164.	192.	275.	43.749,696	21.52	0.112	126.05	0.656
14 0	4 87	1,139,600		1,139,600	280,553,052	164.	192.	251.	39,421,248	21.09	0,11	115 12	0.60
14 0	4 87	1,308,100		1,308,100	336,525,767	164.	190.	260.	40,980,960	18.00	0.10	125.14	0.60
		296,164		296,164	48.674,280	221.	244.	164.	33.444,379	32 77	0.134	108.51	0.44
		350,220		350,220	60,866,730	221.	244.	174.	35,363,874	29.05	0.119	90.02	0.36
		360,356		360,356	58,880,010	221.	244.	163.	33,250,027	27.06	0.11	93.62	0.38
		411.320		411,320	71.686,150	221.	244.	174.	35,465,926	23.91	0.097	76.31	0.32
		394,678		394 678	70,016,730	221.	244.		36,100,630				
9.0	5.25	232,366	4,900	237,266	42,276,200	250.	278.	182	41,312,220	26.87	0.10	156·27	0.56
	5.25, 2.53	372,101	4,864	376,965	71,046,042	250.	278.	188.	43,696,807	29.38	0.105	147.43	0.53
9.0	5 25, 2.53	488,536	4,238	492,774	93,227,568	250.	278.	189.	43,843.624	19.46	0.07	123·77	0.445
9.0	5.25, 2.53	490,168	3,515	493,683	92,412,000	250.	278.	185.	43,400,111	28.49	0.102	138·59	0.497
9.5		459,358	470	459,828	87,424,800	250.	278.	193.	44,080,840	25.39	0.091	130.75	0.47
7.8	4.70, 5.17	404,021	374	404,395	76,640,143	239.5	241.4	185.	37,157.203	28.88	0.119	184·60	0.765
7.6	4 90, 5.15	522,985	457	523,443	102,801,607	243.9	245.7	196·	40,176,620	21.15	0.088	186.45	0.71
10.1	4.50, 4.90	594,160	744	617,904	120,844,126	247 41	249.6	195·	36,679,202	27.20	0.109	185.56	0.74
7.3	4 50	707,514	463	707,977	132,807,571	248.8	250.2	187·	39,145,128	22.95	0.091	167.84	0.62
7.0	4 53	770,750	548	771,298	156,644,383	253.5	255.1	203·	43,188,329	19.68	0.076	153.05	0·59

TABLE III.—CONSUM

Statistics for the Years 1888-1892. In Form Adopted by

Compiled by the Senior E

Name of Place.	Year.	1. Estimated Population. Total at date.	2. Estimated Population. On line of pipe.	3. Estimated Population. Supplied at date.	5. Quantity used through domestic meters, gallons.	6. Quan used th manufac mete gallo
Boston, Cochituate Works.	1888	378,600		370,000	393,970,000	2,477,2
" "	1889	387,600		380,000	308,040,000	2,655,3
" "	1890	410,600		405,000	318,840,000	2,978,8
" "	1891	422,100		417,000	3,717,945,000	
" "	1892	433,600			4,108,687,500	
Boston, Mystic Works.	1888	108,000		106,000	3,804,318	461,9
" "	1889	111,200		110,000	4,432,000	480,8
" "	1890	117,700		115,000	6,978,400	554,1
" "	1891	121,200		120,000	673,625,900	
" "	1892	124,800			681,577,500	
Burlington	1888	16,000	14,200	13,700	28,253,200	12,9
"	1889	16,000	14,200	13,700	28,331,402	12,8
"	1890	14,590	14,300	14,000	40,957,500	7,0
"	1891	14,450	14,450	14,150	42,133,875	16,1
"	1892					
Fall River	1888	63,696		60,524		
"	1889	68,774		64,000		
"	1890	74,918		69,000		
"	1891	77,329		71,000		
"	1892	83,026		76,000		
Fitchburg	1888	22,000	20,000	15,500	21,744,057	119,6
"	1889	23,000	20,500	16,740	25,677,743	120,4
"	1890	22,007	19,500	16,000	32,768,371	125,0
"	1891	25,000	22,500	17,000	36,228,000	130,1
"	1892	27,500	25,000	21,000	43,617,000	153,6
Holyoke	1888	33,229	32,729	32,429	178,684,500	
"	1889	34,728	34,228	33,928	192,327,700	
"	1890	35,637	35,137	34,837	197,396,250	
"	1891	36,114	35,614	35,114	208,645,500	
"	1892	38,500	38,000	37,500	209,300,250	
Lynn	1888	55,529		50,256	100,308,150	
"	1889	58,882		56,099	121,930,400	
"	1890	59,400		56,430	124,620,000	
"	1891	62,500		59,400	149,275,000	
"	1892	67,000		64,700	209,245,000	
New Bedford	1888	37,500	33,550	31,826	4,286,394	157,9
"	1889	40,000	36,500	34,000	8,190,944	259,6
"	1890	41,500	37,500	35,740	9,088,313	359,0
"	1891	45,000	40,000	38,500	9,914,205	327,8
"	1892	50,000	45,000	41,776	9,588,881	272,5
New London	1888	13,500	11,750	9,775		
"	1889	14,000	12,500	10,500		
"	1890					
"	1891					
"	1892	14,000	13,800	12,600		
Newton	1888	23,500	22,500	22,000	102,100,000	12,8
"	1889	23,500	22,500	22,000	129,500,000	17,1
"	1890	25,000	24,000	23,590	139,200,000	22,1
"	1891	26,000	25,200	24,700	151,800,000	13,3
"	1892	27,300	26,500	26,000	175,000,000	15,1
Springfield	1888	41,000	35,300	30,500	116 698,858	
"	1889	44,000	36,500	31,500	147,226,233	
"	1890	44,164	37,000	32,000	169,646,406	
"	1891	45,460	38,500	33,500	189,304,641	
"	1892	49,300	42,000	36,500	212,454,760	
Taunton	1888		22,800	20,100	34,132,833	67,04
"	1889		22,900	20,700	32,432,212	81,96
"	1890		23,000	20,800	37,276,077	82,84
"	1891	25,448	23,210	21,320	42,500,703	96,25
"	1892		23,380	21,870	51,305,470	90,80
Troy	1888	60,000	51,368	45,700	46,329,573	
"	1889	60,000	51,368	46,000	280,753,287	
"	1890	60,650	56,000	56,000	336,735,537	
"	1891	65,000	60,000	58,000	324,220,168	
"	1892	65,000	60,000	58,000	357,121,000	
Waltham	1888	18,000	17,000	16,200	11,619,289	
"	1889	19,800	18,600	18,000	12,076,335	
"	1890	19,000	18,200	18,000	14,043,675	
"	1891	20,000	19,000	18,800	21,041,072	
"	1892	21,000	20,000	19,500	24,929,792	
Ware	1888	7,800	6,500	4,500	4,078,585	6,362
"	1889	7,500	6,500	4,500	8,881,625	4,990
"	1890	7,315	6,500	4,500	14,553,314	5,500
"	1891	7,315	6,550	5,000	13,863,032	7,183
"	1892				25,733,913	
Wellesley	1888	3,400	2,500	1,726	7,055,937	96
"	1889	3,500	2,650	2,450	10,277,198	136
"	1890	3,590	3,340	2,650	12,3.2,637	375
"	1891	3,590	3,379	2,795	14,467,238	459
"	1892	3,625	3,400	2,866	10,447,125	1,029
Woonsocket	1888	20,000	16,750	10,500	29,725,947	5,610
"	1889	20,000	17,150	11,000	29,675,832	8,738
"	1890	22,000	20,000	15,000	70,393,244	
"	1891	22,000	20,000	15,000	79,217,992	
"	1892	24,500	22,000	18,000		

6.		7.	8.	9.	10.
Quantity sed through nanufacturing meters, gallons.	Percentage of total consumption, metered.	Average daily consumption, in gallons	Gallons per day.		
			Each inhabitant.	Each consumer	Each tap.
2,477,262,500	23.6	33,310,700	88.	90.	585
2,655,322,000	25.3	32,070,000	82.7	84.4	545
2,978,872,500	26.7	33,871,700	82.5	84.7	558
,000	27.	37,686,900	89.3	90.4	599
,500	27.2	41,312,400	95.3		635
461,977,300	15.4	8,258,400	76.5	77.9	469
480,842,000	17.1	7,830,500	70.4	71.2	423
554,178,300	18.5	8,301,400	70.6	72.1	425
,900	20.4	9,055,200	74.7	75.5	440
,500	19.	9,810,800	78.6		454
12,953,250	16.4	696,877	45.	50.	286
12,877,910	16.	705,639	44.	50.	281
7,041,000	17.	765,400	52.	55.	300
16,170,000	19.5	817,809	55.	58.	314
	20.	789,289			
	38.1	1,768,524	27.8	29.2	401
	39.5	1,877,937	27.3	29.3	400
	38.5	2,136,182	28.5	31.0	429
	40 2	2,355,700	30.5	33.2	449
	43.	2,285,948	27.5	30.1	414
119,600,700	29.8	1,300,000	57.	84·	529
120,435,100	30.4	1,316,000	56.	78·	490
125,000,000	31.1	1,425,000	65.	89·	491
130,193,000	29.2	2,355,700	62.	88·	502
153,681,000	27.	2,285,948	72.	95·	593
500		2,381,190	72.	73·	1,042
700		2,482,393	72.	73·	1,012
250		2,548,045	71.5	73·	986
500		2,717,721	75	77·	1,011
250	19.2	2,979,689	77.7	79.4	1,054
150		2,474,556		49.2	
400	13.6	2,450,413	41.6	43.7	284
000	12.8	2,656,690	44.7	47.1	289
000		3,131,352		52.7	
000	16.1	3,549,293		54.8	
157,904,641	13.35	3,319,707	88.	104.	574
259,690,821	20.6	3,554,908	89.	105.	582
359,034,506	24.6	4,066,200	98.	114.	636
327,846,754	22.65	4,145,648	92.	108.	615
272,550,329	17.6	4,393,320	88.	105.	616
		1,342,000	99.2	137.	731
		1,378,000	98.4	131.	713
		1,169,171	83.5	92.	495
12,800,000	44.9	708,491	29.8	31.5	177
17,130,000	47.1	853,435	36.	38.	203
22,100,000	44.8	985,396	39.4	42.	222
13,300,000	42 4	1,067,294	41.	43.2	227
15,100,000	40.4	1,288,000	47.	49.5	258
358	8.	4,000,000	97.	131.	777
233	10.	4,000,000	91.	127.	728
106	11.62	4,000,000	90.	125.	693
341	12.93	4,010,986	88.	120.	649
760	13.57	4,289,013	87.	102.	646
67,043,045	36.8	751,882	32.	37.	248
81,966,873	40.7	769,551	32.	37.	244
82,843,716	41.3	796,716	31.	38.	245
96,256,675	37.1	1,022,211	40.	48.	302
90,802,061	38.3	1,012,168	40.	46.	286
73	1.5	8,000,000	137.	160.	1,523
87	9.9	7,839,835	130.	170.	1,369
37	12.2	7,608,468	125.	136.	1,315
68	10.9	8,173,513	125.	141.	1,372
00	11.1	8,240,788	127.	142.	1,122
89	6.	534,063	29.7	32.9	243
35	6.	569,342	29.	31.6	242
75	6.1	625,779	33.	34.4	251
72	7.5	768,638	38.3	40.9	290
92	7.4	921,998	43.9	47.3	323
6,362,373		133,359	17.	31.	365
4,990,906		166,758	22.	37.	412
5,500,222		161,096	22.	36	349
7,183,971		196,400	27.	39.	375
13		191,576			
96,330	19.	138,156	40.	80.	335
136,725	16.	194,646	55.	79.	408
375,292	13.6	255,418	70.	91.	507
459,000	16.1	253,183	70.	91.	464
1,029,247	13.1	239,520	66.	83.	410
5,610,670	35.9	269,528	13.	25.7	313
8,738,836	37.7	302,310	15.	27.5	307
44	58.5	326,455	15.	21.8	293
92	52.5	360,538	16.4	24.	282
		423,562	17.3	23 5	301

TABLE II.—FINAN

Statistics for the Years 1888–1892. In Form Adopted by th

Compiled by the Senior E

Name of Place.	Year.	Receipts From Consumers.					Receipts From Public Funds			
		A.	B.	C.	D.	E.	F.	G.	H.	I.
		Rates, Domestic.	Rates, Manufacturing.	Net Receipts for Water.	Miscellaneous Receipts.	Total Receipts.	Hydrants.	Fountains.	Street Watering.	Public Buildings.
		$	$	$	$	$	$	$	$	$
*Boston Cochituate Works.	1888	618,491.50	581,547.11	1,200,038.61	39,588,54	1,239,627.15	86,976.00			30,371.3
" "	1889	624,825.51	629,613.84	1,254,439 35	27,726.66	1,282,166 01	74,250.00			29,048.9
" "	1890									
" "	1891									
" "	1892									
*Boston Mystic Works.	1888	185,158.57	105,937.47	291,096.04	1,960.41	293 056.45	8,778.00		3,702.13	3,061.0
" "	1889	191,546.92	108,724.41	300,271.33	2.047.68	302,318.91	9,410.00		3,398.39	4,117.5
" "	1890									
" "	1891									
" "	1892									
Burlington	1888	28,853.45	2,652.55	31,506.00			1,200 00	325 00		69 0
"	1889	28,629 22	2,400.00	31,029.22			1,200.00	325.00		75 0
"	1890	28,926.46	2,763.33	31,689.79			1,200 00	325 00		75.0
"	1891	30,892.42	3,096.77	33,989.19			1,200 00	325.00		75 0
"	1892									
Fall River	1888			84,931.79	6,954.02					
"	1889			91.741.83	8,224.69					
"	1890			100,141.23	8.793.68					
"	1891			112,930.77	6,926.15					
"	1892			116,369.02	6,634.78					
Fitchburg	1888	26,298.18	7.973.38	34,271 56			12,800 00	900 00	800.00	2,500.0
"	1889	29,117.26	8,390.58	37,507.84			12,800.00	900.00	800 00	1.500.0
"	1890	32,568.20	9,019.07	41,587.27			12,800.00	900.00	800 00	1,500 0
"	1891	33,621.54	9,303.79	42,925.33			12,800 00	900.00	800.00	1 500 0
"	1892	40.408.37	11,568.12	51,976.49			12,800 00	900.00	800.00	1,500.0
Holyoke	1888			55,677.03			3,000 00			277.8
"	1889			61,738.70			3,000.00			292.6
"	1890			63,120.55			3.000.00			298 4
"	1891			62,607.76			3,000.00			251.1
"	1892			63,923.21			3,000.00			251.1
Lynn	1888									
"	1889									
"	1890									
"	1891	116,159.75	29,855.44	146,015 19	1,793.02	147,808.21				
"	1892	127,305 38	39,040.31	166,345.69	2,003.11	168,348.80				
New Bedford	1888	44,168.18	5,367.36	49,535.54	348.00	49,883.54				
"	1889	49,226.75	6,519.14	55,745.89	273.50	56.019.39				
"	1890	52,777.08	9.704.45	62,481.53	205.64	62,687.17				
"	1891	57,536.13	8,853.79	66,389.92	368.75	66,758.67				
"	1892	64,068.87	7,803.68	71,872.55	354.45	72,227 00				
New London	1888			23,575.01						
"	1889			24,605.22						
"	1890			26,767.52						
"	1891			28,313.95						
"	1892			30,702.76						
Newton	1888	46,478 92	2,213.29	48,692.21	3,747.05	52,439.26	10,800.00	1,091.69	2,000.00	1,449.5
"	1889	48,671.51	2,625.27	51,296 78	5,519.39	56,816.17	11,320.00	1,045.23	1,797.84	1,147.8
"	1890	51,930.99	3,009.03	54,940.02	4.071.47	59,011.49	12,240.00	1,172.50	1,748.79	1,063.8
"	1891	55,645.29	2.264.92	57,910.21	5,421 05	63,331.26	12,840.00	1,325.00	2,100.00	1,025 7
"	1892	61,849.66	2,419 15	64,268.81	4,849 72	69,118.73	13,000.00	1,438.84	2,200.00	1,115.7
Plymouth	1888	14,294.28	1,551.17	15,845.45	252.03	16,097.48				
"	1889	15,472.57	572.72	16,045.29	20.11	16,065.40				
"	1890	15,022.79	1,106.09	16,128.88	32 49	16,161.37				
"	1891	15,518.64	1,114.50	16,633.14	6 50	16,639.64				
"	1892	16,285.05	1,326.00	17,611.05	550.87	18,161.92				
Springfield	1888			112,581 21	16,882 66	129,463.87	11,140.00	1,730.00		2,865.1
"	1889			122.541.52	15,915.90	138.457.42	12,020.00	1,818.90		3,047.4
"	1890			149,913.27	19.946.95	169,860.22	12,160 00	1,789.00		3,240.6
"	1891			136,985.11	18,462 79	155,447.90	12,580.00	1,878.10		3,351.7
"	1892			146,349.09	25,239.65	171,588.74	13,340.00	2,034.10		3,366 2
Taunton	1888	31,231.23	6,553 17	37,814.40	161.68	37,976.08	6,000 00	881.47	800.00	748 5
"	1889	33,331.52	7,398 76	40,730.28	39.20	40,769 48	5,000.00	1,678 37	800.00	832.6
"	1890	31,877.02	8 780.59	40,657.61	20.50	40,678.11	4,000.00	1,079 59	800 00	427.0
"	1891	32,632.06	10,421.25	43,053.31	38.88	43,092.19	4,000.00	1,070.73	800.00	523.4
"	1892	34,685.62	9,803.52	44,487.14	196.38	44,683.52	5,500 00	1,156 02	800.00	511.2
Troy	1888			82,633.25	8,869.81	91,503.06				
"	1889	63,064.03	14,140.77	77,204.80	8,305.89	85,510.69				
"	1890	63,484.42	22,176.39	85,660.81	6,201.28	91,862 09				
"	1891	65,339.97	19,962.00	85,301.97	6,058.06	91,360.03				
"	1892	66,480 02	18,861.53	85,341.55	8.169.31	93,510.86				
Waltham	1888	33,675.86	2,982.61		2,963.00		3,750.00	340 00		450.0
"	1889	36,292.49	3,101.76		4,297.78		3,960 00	373.00		472.0
"	1890	40,553.50	3,661.99		1,594.89		4,040.83	436 00		559.5
"	1891	43,669.43	5,148.79		1,093.86		4,270.00	460 00		606.5
"	1892	46,395.44	5,363.01		1,979.74		4,380.00	784.08		644.6
Ware	1888			5,099.57		5,099.57	1,100.00	133.54		128.3
"	1889			5,708.36	94.64	5 803.00	1,000 00	195 89		133.0
"	1890			5,603.75	206.79	5 810.54	1,000.00	274 22		133.0
"	1891			6,199 63	241.39	6,441.02	1,000.00	310.65		133 0
"	1892			7,828.32	321 38	8,149.70	1,000.00	313.00		133.0
Wellesley	1888	5,209.60	85.00	5,224.60	104.88	5,330.48	2,140.00	40 .00	200.00	50.0
"	1889	6,370.94	89 18	6,460.12	87.60	6,547.72	2,520.00	400.00	200.00	50.0
"	1890	8,786.31	85.00	8,871.31	487.16	9,358.47	2,680.00	400 00	200.00	50 0
"	1891	8,629.70	199.70	8,829 40	601.63	9,431.03	3,100 00	400.00	200.00	62.3
"	1892	9,420.26	411.99	9,832 25	309.62	10,141.87	3,060.00	400.00	200.00	84.4
*Woonsocket	1888	10,118.26	1,334.75	11,453.01	159.30	11,612.31	8,096.25	1,173.68	305.81	126.3
"	1889	10,720.53	1,352.86	12.073 39	375.31	12,448.70	8,523.75	1,297.68	431.99	184.8
"	1890	16,849.02	1,943.68	18,792.70	199.53	18,992.23	12,130.00	1,462.27	895.70	280.2
"	1891	18,473.73	2,823.55	21,297.28	195.84	21,493.12	12,835.00	1,214 96	1,107.17	292.7
"	1892	20,604.31	4,356.27	24,960.58	116.56	25,077.14	13,277.50	1,300.67	2,496.23	386.5

*Woonsocket figures for 1888 are for 8 months only.
Boston " " 1891 " 13 "

oted by the New England Water Works Association.

he Senior Editor.

Public Funds.			K.	AA.	BB.				DD.
	I.	J.							
t r-	Public Buildings.	General Appropriation or Miscellaneous.	Gross Receipts.	Management and Repairs.	Interest on Bonds.	Refunded Water Rates.	Total Maintenance.	Bonds Paid.	Balance.
	$	$	$	$	$	$	$		$
	30,371.31		1,356,974.56	383,638.16	735.791.40	515.27	1,119,944.83		237.029.63
	29,048.95		1,385,464.96	345,986.88	755.268.27	1,873.33	1,103,128.48		282.336.48
			1,382,422 53	381,147.10	765,079 10	1,293 24	1,147,519.44		234,903.09
			1,946,446.16	398 755.92					
			1,518,813 98	392,762.21	810.981.63	963.05			
13	3,061.05		308.597.63	162 086.42	41,992.50	89,862.73			14,655.98
39	4,117.57		319,244 87	125 660.21	41,642 50	98,131.52			53,810.64
			332,634 02	144.184.44	42,357.50	101,985.84			44,106.24
				174 421.92					
			395,792 47	129,354.49	19 257.50	137,749.55			
	69 00		33,100 00	13,766.73	11,191.00		24,957.73		8,142.27
	75 00		32,629.22	15,463.58	11,192.00		26,655.58		5,973.64
	75.00	1,253.61	34,543.40	16,418.20	11,192.00				6,933.20
	75 00	3,545.28	39,134.47	20.055.53	11,263.00				7,815.94
		30,000 00	121,885 81	32,787.20	96,970.00				
		30.000 00	129,966 52	36 984 98	96.640.00				
		20,500.00	128,434 91	37,206 79	97,375.00				
		15.000 00	134,856.92	40.378.24	95,705.00				
		12,800.00	135,503.80	40 933.22	97,420 00				
00	2,500.00	51 85	50,323.41	12.768.17	27,930.00				9,625.24
00	1.500.00		53,507.84	14,129 20	27.930.00				11,448.64
00	1,500 00		57,587.27	8 142.49	27,930.00				21,514.78
00	1 500 00		58 925.33	9,841.46	29,930.00				19,153.87
00	1,500.00	22,839 22	90.875.71	17,653.67	23,930 00				49,292.04
	277.80	8,809.99	67,764 82	10,339.59	15.000.00		25,339.59		
	292.60	20,080.01	85,111 31	9,209.23	15,000.00		24,209.23		
	298 40	17,038.06	83,457.01	9 908.89	15,000.00		24,908.89		
	251.12	9,192 70	75,051 58	12,639.30	15,000.00		27,639.30		
	251.12	8.038.38	75.212.71	11,353 84	15,000.00		26,353.84		
			140,720.59	26 164.19	67,308.50				
			146,582.21	21,248.94	73,462.80		95,747.34		
			162 617.89	22 107.36	81,262.37		103,444.03		
		20,000 00	167,808.21	30.872.86	82,547 44		113,420.30		
		20,000.00	188,348.80	28,126.14	69,893.76		98,019.91		90,328.89
		88,300.00	138,183.54	31,248.84	46,300.00			30,000.00	30,634 70
		88,400.00	144 419.39	27.091.50	46,400.00			30,000.00	40,927.89
		84,900.00	147.587 17	30,701.31	42,900.00			30,000.00	43,985.86
		83,000.00	149,758.67	53,824.48	41,000.00			30,000.00	24,934.19
		81,700.00	153,927.00	37.761.70	39,100.00			30,000.00	46,465.30
				7,174.27	17,500.00		24,674.27		
				4,972.90	17,500.00		22,472.90		
				5,338.78	22,140.00		27,478.78		
				6 085.20	22,140.00		28,225.20		
		6,500.00		5 341.82	22,140.00		27,481.82		
00	1,449.57		67,780.52	13,938.33	42,900.00	258.56			
34	1,147.89	351.81	72 478.94	15,420.78	59,600.00	408.01			
79	1,063.80	89.99	75.326.57	14,794.36	66,840.00	425 62			
00	1,025 77	125.83	80.807.86	13,975 02	74,160.59	293.98			
00	1,115.72	542.73	87.416.02	14 887.98	79,500 00	410.89			
			16,097 48	5,532.20	6,130.00			1,300.00	3,135.28
			16,065.40	4,198.07	6,318.00			1,300.00	4,249.33
			16,161 37	4,831.99	5,820 00			4,100.00	1,409.38
			16 639.64	5,234.82	5,236.00			4,100.00	2,068.82
			18,161.92	6,260.75	5,072.00			4,100.00	2,729.17
	2,865.10		145,198 97	43,973.82	82,000.00				19,225.15
	3,047.45	14,000.00	169,343.77	31,953.65	82,000.00				
	3,240.60	10.000.00		34,799.51	82,000.00				
	3,351.70	14,960.41		28,073,27	86,375.00				
	3,366 25		190 329.09	41,978.21	86,375.00				
0	748 50		46,406.05	14.167.95	26,123.00		40,290.95		6,115.10
0	832.66		48 480.51	12.618.33	26,423.00		39,041.33		9,439.18
0	427.00		46,984 70	13,060.84	27,108.00		40,168.84		6,815,86
0	528.49		49.491.41	13.2 4.97	28,823.00		42,117.97		7,373.44
0	511.26		50.659.80	13.884.49	30,343.00		44,227.49		6,423.31
			91,503.06	53,057.18	16,610.00		69,667.18		21,835.88
			85,510.69	54.674.64	15,847.50		70,522.14		14,988.55
			91,862.09	37,873.67	15,422.50		53,296 17		38,565.92
			91,360.03	43,122.75	14,822.50		57,945.25		33,412.78
			93,510.86	59,368.06	13,885.00		73,253.06		20,257.80
	450.00		44,162.13	18.593.22	18,800.00				6,772.91
	472.00		48,497 03	12.471.96	19,680.00				16,345.07
	559.50		50,846.71	10,029.97	18,746.65				22,070.09
	606.50		55,248.58	12,866.62	19,430.00				22,951.96
	644.68		59,546.95	12,454 31	18,330.00				28,662.64
	128.33		6,461.44	2,501.53	2,780.00		5,281.53		
	133.00	131.70	7.263.59	2,797.28	2,682.00		5,479.28		
	133.00	44.25	7,235.01	2,952.18	2,560.04		5,512.22		
	133 00	99.15	7,983.82	3,027.27	2,442.99		5,470.26		
	133.00	319.21	9,914 91	3,145.43	2,305.24		5,450.67		
0	50.00	1,600.00	9,720.48	2,971.08	3,640.00				3,109.40
0	50.00	1,100.00	10,817.72	3,934.56	6,540.00				
0	50 00	360.00	13,048 47	4,938.98	6,600.00				
0	62.39		13,193.42	4,772.95	7,320.00				
0	84.43		13.886.30	3,870.85	7,560.00				
1	126.38	4,399.29	25,713.72	4,889.87	8.888.76				11,935.09
9	184.84	7,857.68	30,744.64	6,849.74	12,327.77				11,567.13
0	280.25	8,356.79	42,117.24	8,331.07	14,092 80				19,693.37
7	292.79	8,949.27	44,992.31	8,137.53	14,153.57				22,701.21
3	386.51	8,869.24	51,407.29	8,943.08	15,132.71				27,331.50

TABLE IV—DIST

Statistics relating to MAIN PIPES for the Years 1888—1892. In For

Compiled by the Ser

Name of Place.	Year.	1. Kind of pipe used.	2. Size of distribution pipe in inches.	3. Distribution extended during year.	4. Distribution discontinued during year.	5. Total length of distribution pipe in use. Miles.	Cost of re-
Boston	1888	Cast iron.	4" to 48"	122,624	16,030	456.7	$
Cochituate Works.	1889	"	"	133,754	12,081	479.7	
"	1890	"	"	105,835	5,725	498.7	
"	1891	"	"	111,178	5,258	519.0	
"	1892	"	"	94,460	4,270	536.0	
Boston	1888	Cast iron and wrought iron cement lined.	3" to 30"	35,809	12,641	141.5	
Mystic Works.	1889	"	"	42,023	10,425	147.5	
"	1890	"	"	39,923	3,898	152.3	
"	1891	"	"	53,768	22,357	158.	
"	1892	"	"			160.4	
Burlington	1888	Wrought iron, cement lined;	½" to 10"	6,387	3,714	30.09	
"	1889	"	"	4,902	3,224	30.4	
"	1890	wrought iron;	½" to 16"	860	420	30.4	
"	1891	cast iron.	"	10,837	6,074	31.4	
"	1892					31.6	
Fall River	1888	Cast iron.	6" to 24"	8,692		61.4	
"	1889	"	"	6,300		62 5	
"	1890	"	"	5,540		63.6	
"	1891	"	"	6,267		64.8	
"	1892	"	"	9,590		66.6	
Fitchburg	1888		2—16	12,346		38.14	Cemer CastIr
"	1889	Wrought iron cement	2—16	8,591		41.1	
"	1890	lined and cast	2—16	14,366		43 7	
"	1891	iron.	2—16	6,793		42.9	
"	1892		2—30	36,815		50.2	
Holyoke	1888		½" to 20"	16,579	2,917	45.	
"	1889	Wrought iron	"	8,193	108	46.5	
"	1890	and	"	13,674		49.2	
"	1891	Cast iron.	"	6,836		50 5	
"	1892		"	7,040		51.8	
Lynn	1888		4—16	9,389		82.9	
"	1889	Wrought iron cement	4—16	22,877		87.3	
"	1890	lined and cast	"	24,811		92.	
"	1891	iron.	"	35,554		98.7	
"	1892		2—20	23,944		103.2	
New Bedford	1888		4" to 30"	25,469	1,586	59.2	
"	1889	Wrought iron cement	"	15,685	6,049	61.1	
"	1890	lined and cast	"	12,715	4,901	62.5	
"	1891	iron.	"	16,018	3,657	64.9	
"	1892		"	20,689	866	68.6	
New London	1888		4" to 16"	2,392	0	26.2	
"	1889	Wrought iron cement	"	3,744	0	27.	
"	1890	lined and cast	4" to 20"	39,554	4,256	33.6	
"	1891	iron.	"	12,033	737	34.6	
"	1892		"	12,156	2,214	38.2	
Newton	1888	Cast iron.	4" to 20"	19,918		86.8	
"	1889	"	"	17,786		90.2	
"	1890	"	"	17,984		93.6	
"	1891	"	"	45,828		102.0	
"	1892	"	"	14,570		105.0	
Plymouth	1888		2—20	5,531	520	28.2	
"	1889		"	2,578	350	28.7	
"	1890	Wrought iron cement	"	2,636	816	29.0	
"	1891	lined.	"	1,625		29 2	
"	1892		"	14,728		32.	
Springfield	1888	Wrought iron,	1" to 24"	34,919	16,896	85.	
"	1889	wrought iron cement	"	36,821	4,419	91.1	
"	1890	lined,	"	24,185	17,944	92.3	
"	1891	and cast	"	25,916	12,299	94.9	
"	1892	iron.	"	31,207	11,308	98.6	
Taunton	1888	Cast iron.	4—20	1.12 miles		55 9	
"	1889	"	"	3.43 "		59.3	
"	1890	"	"	2.06 "		61.3	
"	1891	"	"	4.72 "		66.1	
"	1892	"	"	1.76 "		67.8	
Troy	1888	Cast iron.	4—30	3,472	150	53.5	
"	1889	"	"	2,727		55.7	
"	1890	"	"	4,473		55.8	
"	1891	"	"	5,221		56.9	
"	1892	"	"	4,535		57.7	
Waltham	1888	Wrought iron cement	2—24	14,148		36.4	
"	1889	lined,		11,250		37.7	
"	1890	cast iron.		6,359		38.9	
"	1891			15,610		41.9	
"	1892			15,305		44.4	
Ware	1888	Cast iron.	4" to 12"	1,845		7.75	
"	1889	"	"	2,853		8.25	
"	1890	"	"	1,048		8.45	
"	1891	"	"	830		8.6	
"	1892	"	"	1,802		9.0	
Wellesley	1888	Cast iron.	4—12	11,316		19.7	
"	1889	"	"	5,308		20.7	
"	1890	"	"	7,436		22.1	
"	1891	"	"	7,659		23.6	
"	1892	"	"	828		23.8	
Woonsocket	1888	Cast iron.	4 -14	6,310		23.5	
"	1889	"	"	11,060		25.6	
"	1890	"	"	10,576		27.6	
"	1891	"	4—20	20,062		31.4	
"	1892	"	4—20	29,650		37.00	

ISTRIBUTION.

Form Adopted by the New England Water Works Association.

e Senior Editor.

6.	7.	8	9.	10.	11.	12.	15.
Cost of repairs per mile.	No of leaks per mile.	Length of distribution less than 4in. Miles	Hydrants. Number added.	Hydrants. Total in use.	Stop gates. Number added.	Stop gates. Total in use.	Range of pressure at center. Day and night.
			118	5,008	249	4,882	35 to 75
			177	5,285	249	5,147	"
			173	5,458	265	5,412	"
			247	5,684	289	5,691	"
			148	5,834	219	5,910	"
		5.5	21	356	71	1,379	
		5.5	43	999	83	1,462	
		5.4	73	1,074	113	1,575	
		5.5	65	1,116	114	1,689	
		5.5	107	1,223	110	1,799	
10.38	.3		1	163	3	266	70 to 85
7.89	.7	5.	1	164	9	275	"
7.89	.5		1	164	11	278	"
3.20	.35		8	172	60	325	"
				172	22	347	
			12	625	11	626	80
			9	634	13	639	80
			8	642	8	647	80
			7	649	12	659	80
			15	664	15	674	80
Cement, 81.20		2.08	12	268	24	254	High service.
CastIron, 2.05							155—160
7.55		2.39	13	281	15	269	Low service.
9.56		1.83	16	297	15	295	75—80
6.22		1.96	19	314	58	353	
5.77		1.96	16	330	32	385	
14.14			14	288	35	333	45—100
4.95			13	301	20	353	"
6.65			14	315	21	374	"
3.91		5.3	19	455	22	396	"
3.66		5.3	15	487	20	418	
			12	602	27	713	54—65
			19	620	39	752	54—65
			38	658	52	801	"
			47	705	53	854	50— 65
			42	747	38	892	"
14.66		1.1	35	445	68	615	31—40
20.00		1.1	15	460	28	643	30—36
17.87		1.2	17	477	26	669	"
22.47		1.1	23	499	32	701	"
9.94		1.1	43	542	41	742	"
55.42	6.3	1.9	1	137	7	113	27—54
21.14	1.8	1.7	2	139	8	121	25—54
15.89	3.	2.	9	148	28	149	45—54
32.16	1.6	2.2	8	156	17	166	45—50
20.08	1.2	2.5	6	162	18	184	45—50
10.73		2.4	42	588	23	427	60
10.77		2.4	21	606	24	451	60
6.95		2.5	36	642	24	475	69
10.08		2.6	34	676	40	515	84
11.72		2.7	28	704	19	534	84
					25	242	
8.44		7.7		83	16	258	
8.44		7.7	2	93	9	267	
6.89		7.7	1	94	6	274	
6.49		9.5	7	101	13	287	
	1.01		29	557	54	777	H. S., 90-120. L. S., 30-35
	.65		44	601	101	878	" "
	1.26		7	608	114	992	" "
	.44		21	629	48	1,040	" "
	.40		38	667	84	1,124	H. S., 80-115. L. S., 30-35
37.0		.8	12	487	9	356	Ordinary, 45—50
24 06		.8	27	514	22	378	Fire, 95—100
21.17		.8	22	536	21	399	
18.10		1.1	40	576	25	424	
19.92		1.2	47	623	7	431	
11.08		.7	29	629	20	913	High Service, 25—75
7.93		.7	34	661	33	1,010	Middle Service, 25—115
4.89		.7	23	684	44	1,054	Low Service, 35—60
2.29		.7	12	696	53	1,107	
7.10		.6	19	717	35	1,142	
8.87			9	197	52	349	55
12.40	.37		3	200	23	372	"
7.50	.05		11	211	27	399	"
26.38	2.05		7	218	37	436	"
25.83	1.4		14	232	39	475	"
	.2		4	56	3	75	85
	.2		3	59	3	78	"
	.2		2	61	2	80	"
	.2		1	62	1	81	"
	.2		3	65	5	86	"
6.25	.6	.4	23	126	11	113	70
2.02	.6	.4	8	134	10	127	"
1.00	.4	.4	11	145	11	138	"
1.65	.5	.4	8	153	12	150	"
1.35	.6	.5	8	161	2	152	"
1.21			15	349	20	263	90—120
3.67	1.19		28	377	24	287	90—115
3.98	.79		26	403	20	307	"
.75	1.26		38	441	48	355	"
2.68			14	455	38	393	"

TABLE V—DISTRI

Statistics relating to SERVICE PIPE for the Years 18

England Water Works

Compiled by the Senior

Name of Place.	Year.	16.	17.	18.	19.	20.
		Service pipe.				
		Kind of pipe used.	Size of pipe in inches.	Length of pipe extended.	Length of pipe discontinued.	Total length now in use,
Boston (Cochituate Works.)	1888	Lead and cast iron.	½" to 4 "	50,974	2,768	
" "	1889		"	58,768	6,817	
" "	1890		½" to 6 "	61,800	4,606	
" "	1891		"	69,859	5,635	
" "	1892			59,807	5,871	
Boston (Mystic Works.)	1888		½" to 2 "	24,155		95.7
" "	1889			18,527		100.7
" "	1890			20,360		1043
" "	1891			28,695		110.1
" "	1892			31,584		
Burlington	1888	Galvanized iron and lead.	½" to 4 "	3,046	116	13.5
"	1889		"	2,538	100	14.
"	1890		"	904	45	14.1
"	1891		"	1,554	74	14.4
"	1892		"			
Fall River	1888	Lead.	½" to 2 "			
"	1889		"			
"	1890		"			
"	1891		"			
"	1892		"			
Fitchburg	1888	Wrought iron, cement lined and cast iron.	¾" to 8 "	15,745		29.8
"	1889		"	13,805		32.4
"	1890		"	10 308		33.7
"	1891		"	6,469		35.6
"	1892		"	13,856		38.2
Holyoke	1888	Cement lined wrou't iron	½" to 4 "	2,240		8.7
"	1889	Rubber " " "	"	3,500		9.3
"	1890	Enam'ld " " "	"	2,940	17	9.8
"	1891	Cast iron.	"	2,977	19	10.3
"	1892		"	2,820		10.9
Lynn	1888	Cement lined, wrought iron; adamanta; galvanized iron.	½" to 6 "	22,110		62.3
"	1889		½" to 6 "	24,079		66.9
"	1890		½" to 10"	27,338		72.1
"	1891		"	31,868	4,275	80.2
"	1892		"	29,212	7,350	85 8.
New Bedford	1888	Lead and cast iron.	½" to 8 "	12,753		34.2
"	1889		"	16,346		37.3
"	1890		"	9,560		39.1
"	1891		"	11,525	265	41.3
"	1892		"	13,850	103	43.9
New London	1888	Lead, wrought iron cement lined, cast iron and galvanized.	½" to 4 "	1,581	129	6.4
"	1889		"	2,213	120	6.8
"	1890		"	3,146	466	7.6
"	1891		"	2,966	110	8.1
"	1892		"	2,575	32	8.6
Newton	1888	Lead, wrought iron and cast iron.	½" to 4 "	10,378		48.2
"	1889		"	12,892		50.7
"	1890		"	11,795		52.9
"	1891		"	14,355		55.6
"	1892		"	15,892		58.6
Plymouth	1888	Wrought iron cement lined and lead.	½" to 1 "			
"	1889		"	500		5.
"	1890		"	242		5.2
"	1891		"	398		5.2
"	1892		"	343		5.3
Springfield	1888		½" to 4 "			
"	1889		½" to 4 "			
"	1890		½" to 4 "			
"	1891		"			
"	1892		½" to 6 "			
Taunton	1888	Cement lined wrought iron.		miles.		
			¾" to 3 "	1.59	129	29.4
"	1889		"	0.94	2,259	30.3
"	1890		¾" to 3 "	1.39		31.7
"	1891			1.26		33.
"	1892			1.29		34.25
Troy	1888	Lead and cast iron.	½" to 6 "			
"	1889					
"	1890					
"	1891					
"	1892					
Waltham	1888	Wrought iron, cement lined and cast iron.	¾" to 6 "	7,978	136	23.4
"	1889			13,723	1,136	25.8
"	1890			11,303	254	27.9
"	1891			9,095	402	29.8
"	1892			8,802	100	31.1
Ware	1888	Wrought iron cement lined.	1" to 2 "			4.35
"	1889					5.15
"	1890					5,77
"	1891					6.43
"	1892					
Wellesley	1888	Wrought iron, cement lined and lead.	½" to 2 "	5,890		
"	1889		"	4,212		
"	1890		"	2,992		
"	1891		"	4,841		
"	1892		½" to 4 "	4,506		9.3
Woonsocket	1888	Lead.	½" to 1½"	1,615		2.6
"	1889		½" to 1½"	1,829		3.
"	1890		½" to 4 "	2,160	12	3.4
"	1891		"	2,073		3.8
"	1892		½" to 6 "	3,013		4.3

TRIBTUION.

1888—1892. In Form Adopted by the New
orks Association.

Senior Editor.

20.	21.	22.	23.	24.	25.	26a.	26b.	27.	28.
	Service taps.				Meters.			Motors and elevators.	
						Now in use			
Total length now in use, miles.	Number added.	Total in use.	Av. length of service.	Average cost of service.	Number added.	Domestic.	Manufact'y.	Number added.	Total in use.
	1,712	56,947		$		3,133			399
	1,863	58,810			342	3,456			426
	1,908	60,718			172	3,627			451
	2,159	62,877			212	3,839			518
	2,197	65,074			73	3,912			
95.7	798	17,607				387			8
100.7	920	18,527				386			27
104.3	993	19,520			5	391			23
110.1	1,036	20,556			15	406			21
	1,032	21,588			29	435			
13.5	105	2,433	31.	11.09	71	409	36	1	8
14.	82	2,513	31.	7.80	80	486	39	2	10
14.13	38	2,549	22.	7.75	59	541	43	6	16
14.41	61	2,609	29.	8.15	70	605	49	1	17
	57	2,666			57				
	215	4,412			197	3,138			13
	286	4,698			290	3,428			13
	282	4,980			289	3,717			13
	267	5,247			258	3,975			14
	279	5,526			277	4,252			12
29.86	253	2,456	62.	9.92	117	571	86	3	18
32.47	228	2,684	60.	13.09	144	695	98	12	30
33.78	221	2,905	46.	13 49	169	792	160	2	32
35.65	198	3,103	33.	9.53	159	1,000	111	5	37
38.27	266	3,369	48.	6.42	156	1,162	105	7	42
8.7	112	2,245	20.	11.94		143		2	56
9.3	178	2,454	20.	10.23		145			39
9.8	146	2,583	20.	14.56	4	149			38
10.3	104	2,687	20.	14 56	6	155			38
10.9	141	2,830	20.	14.56	22	177			38
62.3	433	8,233			18	254			
66.9	373	8,606			70	324			
72.1	573	9,178			16	340			
80.2	556	10,026			43	413			
85 3.	567	10,588			81	494			
				20.42					
34.2	291	5,785	43.8	3 26	8	63	45	1	27
				22.89					
37.3	320	6,104	51.1	12.76	16	68	52	4	31
				19.70					
39.1	290	6,394	33.	1.50	3	76	47		30
				18.10					
41.3	348	6,742	33.		12	83	52	3	33
				15.52					
43.9	397	7,134	35.		9	91	53		31
6.4	87	1,833	18.4	11.88	0	37		5	18
6.8	131	1,934	18.3	10.98	2	39			18
7.6	153	2,087	20.5	13.80	4	43			18
8.1	156	2,235	19.	12.68	7	50		5	23
8.6	128	2,362	18.6	11.26	80	129		2	25
				26.17					
48.2	211	3,978	64.	12.74	145	2,496	10	2	9
				26.50					
50.7	225	4,203	63.6	12.85	209	2,692	16	3	12
				25.66					
52.9	237	4,440	62.9	12.42	311	2,977	18	2	16
				29.42					
55.6	265	4,705	62.4	13.84	284	3,251	18	1	16
				29.14					
58.6	300	5,002	61.8	15.68	321	3,566	21		16
	47	1,364							
5.	25	1,389	20.	5.92					2
5.2	30	1,415	19.3	5.60					2
5.2	43	1,454	19.3	5.39					2
5.3	49	1,500	18.5	3.80					1
	328	5,147			84	556		15	114
	350	5,497			195	751		9	123
	273	5,770			177	928		17	140
	411	6,181			198	1,126			140
	452	6,633			276	1,402		6	146
				17.77					
29.4	160	3,027		7.82	76	795	63		7
				16.11					
30.3	118	3,143		6.71	80	863	85	1	8
				24.14					
31.7	129	3,253		13.54	52	901	89	1	9
				18.43					
33.	137	3,386		9.03	69	968	91	2	13
				24.20					
34.25	138	3,507		15.11	72	1,036	95		13
	203	5,453	3.		11	58		2	100
	214	5,654			165	223			
	132	5,786			3	226			
	174	5,960			9	228			
	324	6,281			9	237			
23.4	167	2,197	48.	11.50	4	27		2	10
25.8	156	2,339	78.	26.3	6	34			10
27.9	167	2,489	74.	40.57	6	40			10
29.8	176	2,650	47.5	30.60	6	46		1	11
31.1	172	2,820	50 6	26.50	13	59			11
4.35	57	350	53 8		49	76	11		3
5.15	40	381	57.5		91	167	11		3
5,77	58	430	56.4		34	199	13	1	4
6.43	60	490	56.4		32	230	14	1	5
	49	542			55	291			
	53	412	111.		17	65	1	1	1
	53	465	79.	22.40	17	82	1		1
	38	503	79.	17.87	10	92	1		1
	43	546	115.	54.48	14	106	1		1
9.3	39	584	122.	21.31	57	160	4	1	2
2.6	100	861	15.8	13.13	87	743	18		1
3.	122	983	15.5	11.18	93	802	25		1
3.4	135	1,117	16.2	10.99	97	897	27	1	2
3.8	148	1,276	14.	9.64	117	1,013	27	1	3
4.3	191	1,467	16.5	11.44	110	1,120	31	2	5

OBITUARY.

AUGUSTUS W. LOCKE.—Died at North Adams, Mass, Sunday May 14th, 1893. Age 46 years. Joined this Association June 13, 1889.

He served in the Navy during the war and after his discharge took up the study of civil engineering. After preliminary work he was appointed assistant engineer on the Hoosac tunnel, serving in that capacity from 1868 to 1876. He served as engineer of maintenance of way on the Troy and Greenfield road until 1878 when he was appointed by Governor Long manager and chief engineer of the Troy and Greenfield road and of the Hoosac tunnel. He held his position until 1887 when the road and tunnel were sold to the Fitchburg Co. When he retired he received public commendation through Governor Robinson for his able management of affairs. He then entered upon private business and at the time of his death was chairman of the special commission of the state board on the abolition of grade crossings. Contributed an article to the Journal of September, 1890, entitled "Notes Made in Holland in 1887."

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